Engineering News

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Electric Hoist of the Hamilton Inclined Railway*

New hoist equipment, with a variety of novel features, has been installed by the Hamilton Mountain Park Co., Ltd., at Hamilton, Ont., making the Hamilton Inclined Railway probably the "largest passenger incline in the world." The difference in elevation between the general level of the city and the plateau back of the bluffs is 325 ft. As the roads leading up the bluffs are necessarily steep, the inclined railway handles a very large and miscellaneous traffic—transporting passengers, freight, automobiles, trolley cars, teams etc. The railway was formerly operated by a steam hoist, but this became inadequate to the requirements of increased speed and safety.

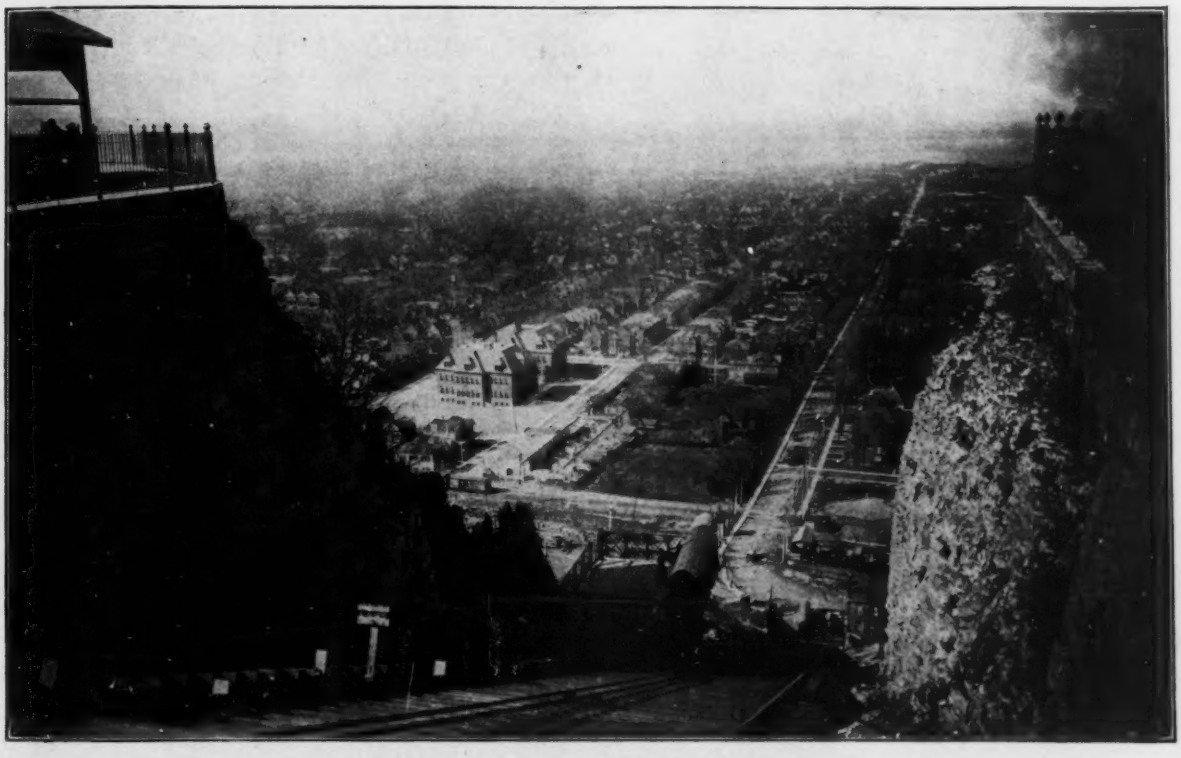

FIG. 1. HAMILTON MOUNTAIN PARK INCLINED RAILWAY AT HAMILTON, ONTARIO

THE INCLINED RAILWAY

Two large platform cars are carried in balance on an inclined double track 800 ft. long, having a grade of 40.3 per cent. The track gage is 12 ft. 1½ in.; each car weighs 30,000 lb.; the average car load is 20,000 lb., the maximum being 30,000 lb.

Two 1⅝-in. ropes, weighing 4.15 lb. per ft., run from each car to a special double fixed-drum double-geared electric incline hoist. One rope is for haulage; the other is a safety cable. The average rope speed during a run is 585 ft. per min. The time of a single trip is 1½ min., and there is a 3-min. rest period between trips.

NEW HOIST

The hoist is located in a house 106 ft. from the knuckle between the incline and the level of the summit. The main rope from the right-hand car is wound over the top of the right-hand hoist drum. The main rope from the left-hand car is wound underneath the left-hand hoist drum. The safety rope from the right-hand car is led over 7-ft. deflecting sheaves to the top of the left-hand drum, while that from the left-hand car is similarly led to the bottom of the right-hand drum.

There are four head sheaves and four deflecting sheaves. The former are arranged vertically so as to carry the

*From information furnished by the Lidgerwood Manufacturing Co., New York City, and the General Electric Co., Schenectady, N. Y.

hoist and safety ropes in a direct line from the cars; the deflecting sheaves are placed horizontally at such an angle that the rope will be led in a direct line either to the top or bottom of the hoist drums, as the case may be. Floating guide sheaves are placed in the rope tunnels between the head sheaves and the hoist drums.

The 12-in. drum shaft is a two-piece steel forging; 52 ft. long and weighing 13,300 lb. The 7-in. intermediate shaft is a single forging 20 ft. long. Two two-section cast-iron barrel drums, 96 in. in diameter and 70 in. on the face, coil 800 ft. of rope. Each drum barrel weighs 16,700 lb. The drums are bolted at one end to a post-brake ring and at the other to a gear.

The cast-steel drum gears are double eight-arm wheels with 122 involute teeth of 12-in. face and 122-in. pitch diameter. The intermediate cast-steel gears have 113 herringbone teeth of 8-in. face and 56½-in. pitch diameter; the intermediate forged pinions have 22 herringbone teeth of 8-in. face and 11-in. pitch diameter.

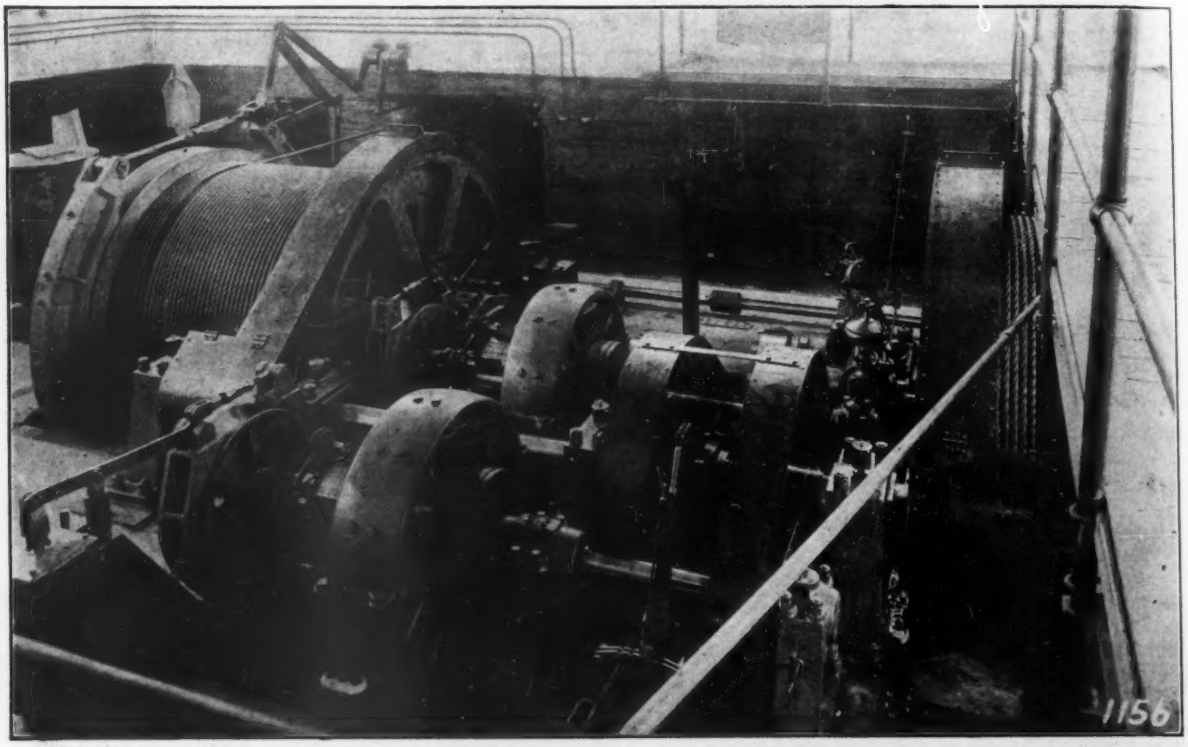

FIG. 2. ELECTRIC HOIST OF THE HAMILTON INCLINED RAILWAY
Showing all of one drum and part of the second, both motors, both solenoid brakes, geared connections, clutches, flyball governor, etc.

DRIVE

A 180-hp. 500-volt 475-585-r.p.m. direct-current motor (designed for the voltage variation of storage-battery service) drives the hoist through two gear reductions with a total ratio of 29.8 to 1. A magnetic-contactor panel gives remote control. To insure continuous service a reserve 180-hp. motor and solenoid are provided and can be substituted in a few minutes.

Power is supplied in the form of three-phase 25-cycle 2200-volt alternating current. For changing this into direct current, a motor-generator set has been installed of sufficient capacity to supply the average demand of the hoist, plus some surplus for charging a storage-battery floating on the line. The generator has a reversed series field throwing load fluctuations on the battery. A small percentage of the load fluctuations falling on the machine will lower its voltage to such an extent that the battery must discharge and furnish the balance of the momentary demand. The regulation is therefore inherent in the design of the machine and is entirely automatic.

OPERATION AND SAFETY APPLIANCES

The operator's cabin is fitted with one electric control and two hand-brake levers. The levers will not be used ordinarily, as the hoist is equipped with solenoid brakes operating on the motor shaft. The hand-brakes therefore need only be used for locking the cars at the top and bottom positions or in an emergency.

In starting a run, the operator releases the drum post brakes by the hand-levers, puts his foot on the small foot-pedal located at the bottom of the master controller and moves the handle of the controller either to the right or the left, as the case may be. The cars start and automatically accelerate to the normal rope speed. At a predetermined point on the incline, the controller handle automatically is turned to such a position that the speed is cut down to one-tenth normal and finally is turned to the off position, setting the solenoid brakes and bringing the cars to rest.

Should the operator become disabled during a run, he will of necessity remove his foot from the foot-pedal, thereby cutting off the current and bringing the cars to rest. In order to move the cars, the operator's foot must be on this pedal.

Should the controller fail to stop the cars, an over-winding device is attached which will shut off the current

and set the solenoid brakes. In case the cars should stop short of their landing positions, owing to the action of the automatic overwinding mechanism, there are two or three points on the controller available so that the operator can bring the cars to their proper positions. Should the speed of the cars exceed the normal by a predetermined amount, an overspeed device (flyball governor) will trip a 570-lb. weight which will set the drum post brakes. The emergency weight can also be tripped manually from the cabin.

The hoist was made by the Lidgerwood Manufacturing Co., of New York City, the electrical machinery and control apparatus by the General Electric Co., of Schenectady, and the storage battery by the Electric Storage Battery Co., of Philadelphia.

The Roman Aqueduct near Tarragona, Spain

By John R. Rippey*

Among the imposing monuments of the Roman period in Spain is the aqueduct of Tarragona. This conduit brought water from the River Gaya, about twenty-two miles, by means of canals and tunnels and by conduits supported on arches and piers.

Although built about 200 B.C., the remains of the masonry aqueduct are in a fine state of preservation today. Figs. 1 and 2 show a portion of the structure extending across a valley about four miles north of Tarragona, and Figs. 3 and 4 show the aqueduct near the city. The valley crossing, shown in Figs. 1 and 2, is composed of two tiers of stone arches. The tower tier of eleven arches is 240 ft. long and 45 ft. high. The upper tier is 712 ft. long and 78 ft. high, thus making a total height of 123 ft. The span of the arches is 22 ft. 6 in. and the piers are 12 ft. square at the base.

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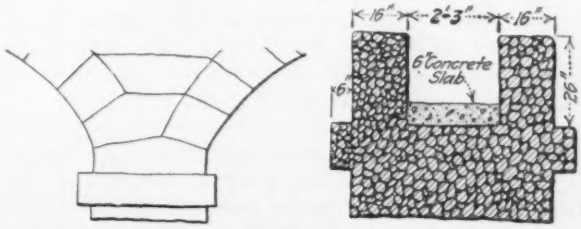
There is apparently no mortar between the 18 voussoirs which make up the arches, arranged as shown in Fig. 5.

From all appearances, this aqueduct was part of an inverted siphon. Very little remains of the waterway. Fig. 6 is a sketch showing the largest portion that remains intact. At several points slabs of concrete were found, which probably formed the top of the siphon. These are about 6 in. thick, and the coarse aggregate is composed of broken brick—the only sign of brick in the structure.

After crossing over the hill shown in Fig. 1, the water was carried on for about three miles and then turned to the

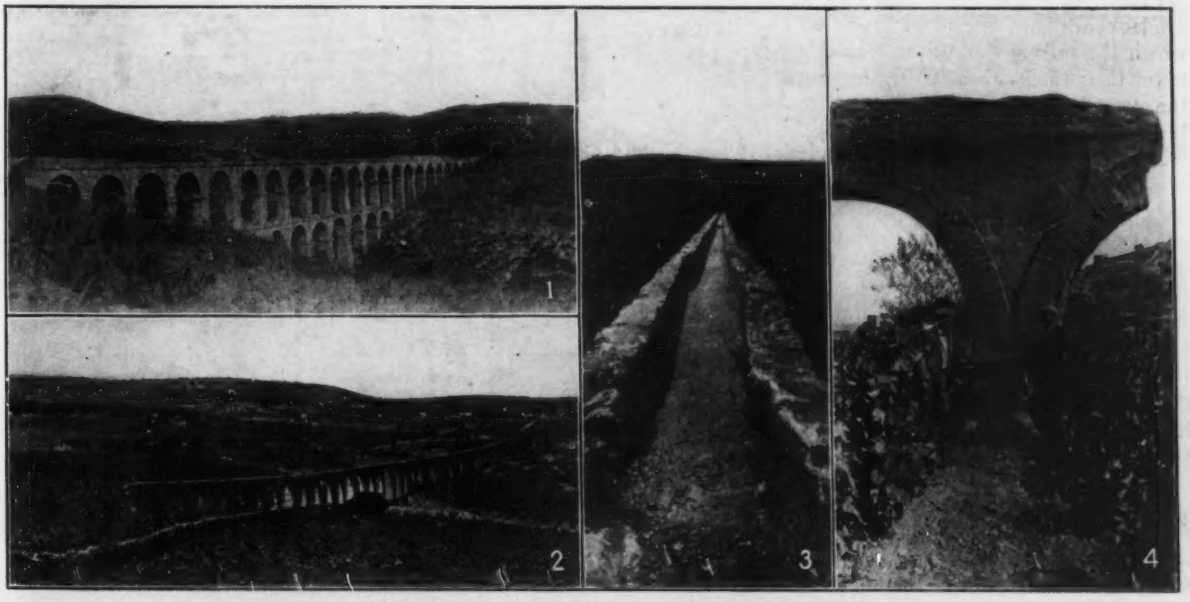


Figs. 5 and 6. Details of Part of Aqueduct Shown in Figs. 1 and 2

Fig. 5—Arrangement of facing stone of arches. Fig. 6—Details of waterway

right and brought into the city, as shown in Figs. 3 and 4. This portion of the aqueduct is built entirely of brick and concrete. Originally, this structure had a total length of about 1200 ft. At present there are only 27 arches standing, each having a span of about 12 ft. and a rise of about 7 ft. The brick used in this work are 12 in. long, 6 in. wide and 1¼ in. thick. The mortar at present is extremely hard.

The Spanish government, realizing the historic value of these aqueducts, has taken steps to preserve them. Before these steps were taken, the people tore down a number of the arches and used the brick for the construction of a number of the neighboring buildings.



Figs. 1 to 4. Views of Two Portions of Tarragona Aqueduct, Spain

Fig. 1—About four miles from the city. Fig. 2—Entering the city. Fig. 3—Waterway of portion shown in Fig. 1. Fig. 4—Detail of brickwork and concrete

Eliminating the Tower Grove Grade Crossing at St. Louis

SYNOPSIS—This complicated crossing involved two intersecting streets (with car tracks), two groups of steam railways, and spur tracks to several industries. The railways were lowered and the streets raised, crossing the tracks by an X-shaped viaduct of reinforced concrete, the two legs of this being of different design. The work was done by the two railways and was carried on without interrupting street traffic. One railway did all its work with its own forces, while the other let contracts for a part of the work.

The elimination of the grade crossings at Tower Grove is the most extensive of the works of this kind at St. Louis, Mo. It involved special problems and difficulties due to the location and the traffic conditions, as may be seen from the plan, Fig. 1.

Tower Grove Ave. and Vandeventer Ave. are two wide intersecting thoroughfares with car tracks and heavy street traffic. At their intersection they are crossed by the numerous tracks of the Missouri Pacific Ry. and the St. Louis & San Francisco Ry. The former has a suburban branch (to Oak Hill) which crosses the latter at grade, and both lines have spurs serving several industries. Each has a station at the crossing. There are about 56 passenger and freight trains on the St. Louis & San Francisco and 70 on the Missouri Pacific passing over the crossing every 24 hours, while the street cars number 519 in the same time.

Some years ago the city passed a grade-separation ordinance under which the railways would have had to lower their tracks about 25 ft., leaving the street level unaltered and throwing the entire expense upon the railways. The industries objected to this, as it would have made it impracticable to serve them by spurs to the ground level, and two of these industries obtained an injunction to prevent the railways from carrying out the work. It was finally decided by the State Supreme Court that, while the city had power to order grade separation, it had no power to compel the work to be done in the most expensive manner and in such a way as to interfere with the facilities enjoyed by the industries.

The railways had shown that a cheaper method would be to carry the streets beneath the tracks. After the above defeat the city prepared a compromise ordinance, under which the railway tracks were to be lowered 12 ft. and the streets elevated 13 ft., and providing that the city would assume the consequential damages to abutting property. This ordinance was accepted by the railways in 1913. Work was commenced early in 1914.

The general plans and the designs for the viaduct were prepared by the railways jointly and approved by the city authorities. The work is under the direction of F. G. Jonah, Chief Engineer of the St. Louis & San Francisco Ry.; C. E. Smith, Assistant Chief Engineer of the Missouri Pacific Ry., and C. E. Talbert, Street Commissioner. The engineers in direct charge are Perry Topping, Assistant Engineer of the St. Louis & San Francisco Ry.; E. S. Wonson, Bridge Engineer of the Missouri Pacific Ry., and L. R. Bowen, Bridge Engineer of the city's Street Department.

The railways pay all construction cost, amounting to about $600,000. This includes industry connections, no part of these being paid for by the industries. The city pays all consequential damages, as before noted, and pays also for the paving. At the north end the elevation for the viaduct approaches made it necessary to raise several buildings to the new grade. As this compelled closing some stores, etc., during the change, the city undertook to pay a portion of the rent in such cases. The street-railway company pays for its temporary track work and the shifting of the tracks, also the cost of track and overhead work across the viaduct. Each of the railway companies assumed the construction of that part of the viaduct over its own tracks, and the dividing line is shown at *A* in Fig. 1.

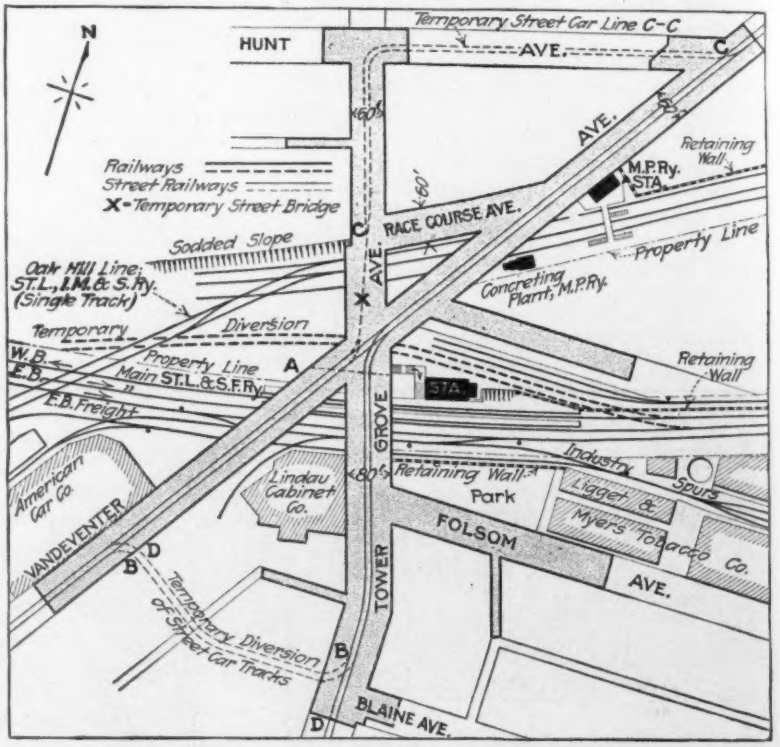


FIG. 1. PLAN OF TRACKS AND VIADUCTS AT THE TOWER GROVE CROSSING, ST. LOUIS, MO.

The layout of the Missouri Pacific Ry. tracks is tentative only. At present it has only the two tracks adjacent to the station, and a single track on the Oak Hill line. Additional tracks will be laid later

The ground is nearly level. The track depression extends for a distance of about 4500 ft. and reaches a maximum depth of about 16 ft. below the original level. The viaducts have a maximum elevation of about 12 ft. above the original level, or 9 ft. to the under side of the floor-beams, giving a clear headway of 22 ft. between rail level and the viaduct. It will be seen by the profile of the St. Louis & San Francisco Ry., Fig. 2, that the original crossing was a summit in the grades, so that the main line is improved by the lowering of the tracks, the grade through the depression being 0.75%. On the other hand, it introduces difficulties in the industry-track connections, for which grades of 1.85 and 3% are required to reach the surface.

The St. Louis & San Francisco Ry. did its grading, including steam-shovel work, with its own forces. The concrete work, including this railway's portion of the viaduct, was let to the Jarrett-Richardson Paving Co., of Springfield, Mo., while the additions and improvements to the passenger station were let to James Stewart & Co., of St. Louis.

During the excavation for the new grade, the two main tracks of the St. Louis & San Francisco Ry. were diverted to pass around the station (at the original level) as shown by the dotted lines in Fig. 1, thus facilitating both the traffic and the construction work. Fig. 3 shows this cut in progress, looking west from the crossing, with the temporary tracks at the right. The Missouri Pacific Ry. branch line is shown shifted beyond its original location and carried by a temporary trestle. With the lowering of the main lines this branch was replaced on its original location, but at the new grade.

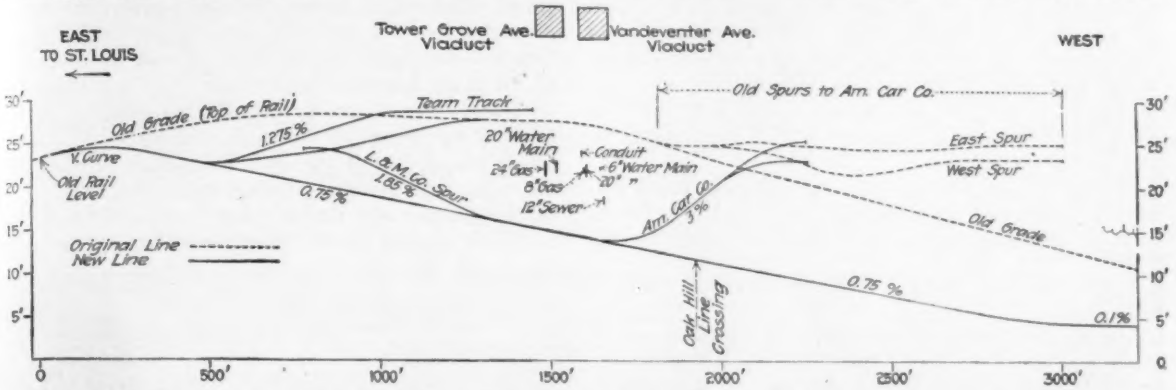


FIG. 2. PROFILE OF THE ST. LOUIS & SAN FRANCISCO RY. AT THE TOWER-GROVE CROSSING, ST. LOUIS, MO.

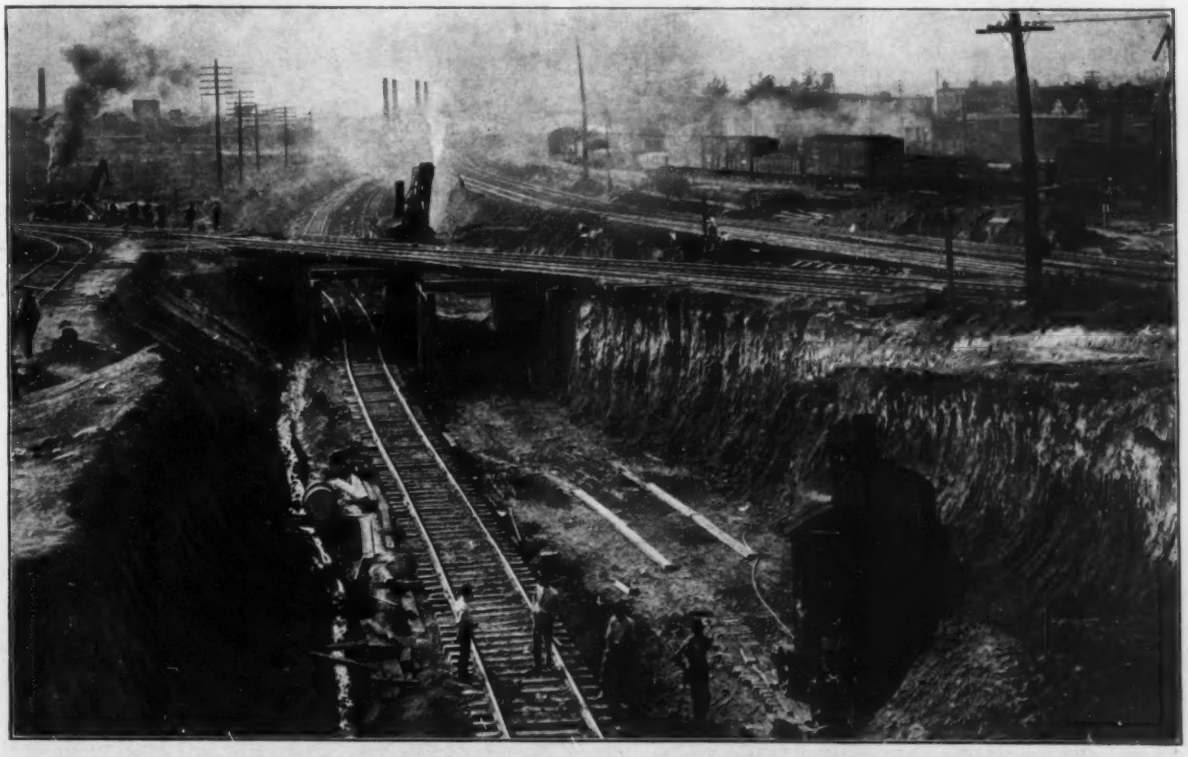


FIG. 3. LOWERING TRACKS AT THE TOWER GROVE CROSSING, ST. LOUIS, MO.

The view is looking west from the Vandeventer Ave. viaduct. The cut is for the St. Louis & San Francisco Ry., traffic being diverted to the two temporary tracks at the right. Beyond the temporary tracks is the main line of the Missouri Pacific Ry., while the line crossing the cut is the Oak Hill branch of the St. Louis, Iron Mountain & Southern Ry.

Fig. 4 shows the work in progress for the St. Louis & San Francisco Ry. at the south end of the Vandeventer Ave. viaduct. The foundations of the viaduct were built in trenches before the general excavation was made. The remaining high-level track at the right is an industry connection (now lowered). Beyond the viaduct may be seen the temporary bridge carrying Tower Grove Ave. across the cut, at the original ground level. At the deepest part, the cut has retaining walls, and on the east side an incline will lead up to a double-track paved team yard at the present ground level, as shown in Fig. 1. The Tower Grove station also will remain at the present level, with stairways to the train platforms and to the viaduct.

On the Missouri Pacific Ry. the restricted space prevented diversion of the tracks, and running tracks had to be kept open while a preliminary cut was made for the mains, and the change of grade of the tracks was complicated by the necessity of maintaining a connection with the Oak Hill branch. The Missouri Pacific Ry. did all its portion of the work (including the viaduct) with its own forces.

In the concreting, the Missouri Pacific Ry. employed a stationary mixer plant with tall elevator tower and inclined chutes to the forms. Smaller intermediate towers supported the long runs of chutes. The plant was placed first near one end of the viaduct and later on the ground in the angle between Vandeventer Ave. and Park Ave. (Fig. 1). The contractor for the concreting on the St. Louis & San Francisco Ry. used a small portable mixer which was shifted as the work progressed. In building the retaining walls and viaduct bents, this outfit stood on the ground and had attached to it a truck with a telescopic elevator tower. For concreting the deck, the

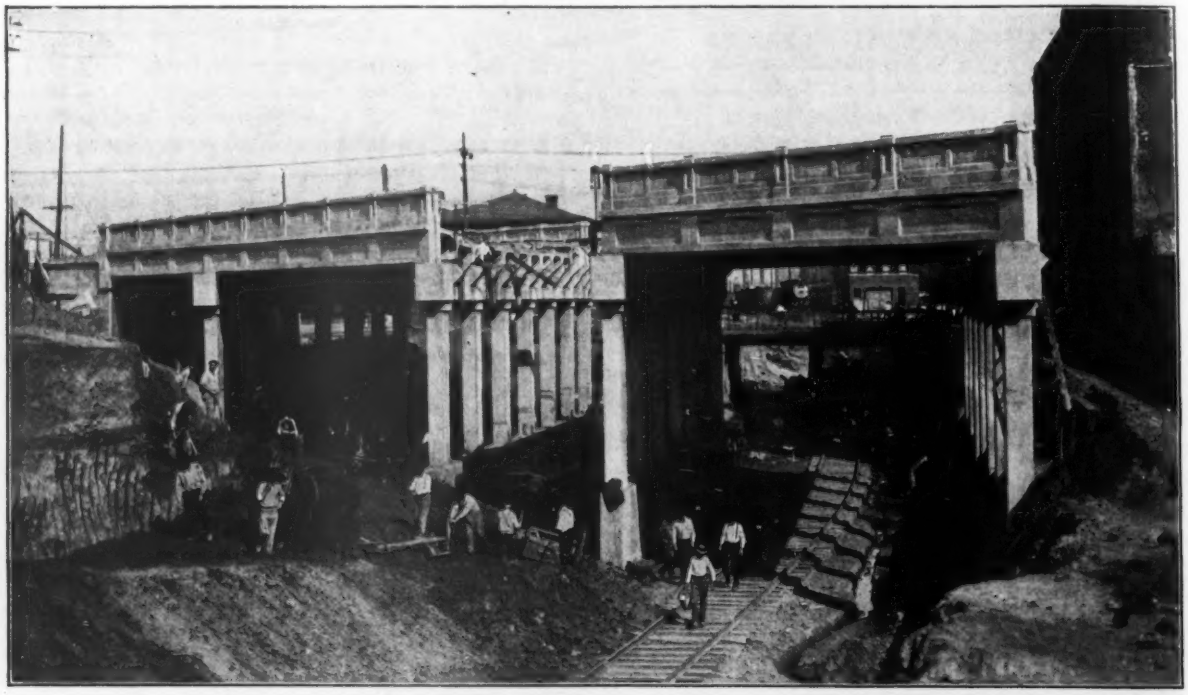


FIG. 4. BUILDING THE VIADUCT OVER TRACK DEPRESSION AT TOWER GROVE, ST. LOUIS, MO.

This is the five-span section of the Vandeventer Ave. viaduct over the St. L. & S. F. Ry. The foundations and bents were built in trenches, and the main excavation was done later, traffic being diverted to a temporary track

first two tracks at the new grade. These were adjacent to the station and had a double-track junction with the single-track Oak Hill branch. The limited traffic on this branch did not warrant the cost of a diversion to secure an overhead crossing of the St. Louis & San Francisco Ry., but an interlocking switch and signal plant will be installed for the protection of traffic over the grade crossing.

South of this double-track cut, trenches were made for the foundations of the viaduct. The bents were built on these and the deck then built, leaving for the future the excavation of the ground to widen the cut for additional tracks. The Tower Grove station (M. P. Ry.) was raised to the level of the viaduct approach and placed against the sidewalk line, as shown in Fig. 1. Later, a bridge will extend from the station across the tracks, with stairs to the train platforms. The excavation of the cut was interfered with by sewers and water and gas portable mixer was placed on the upper level and discharged the concrete directly into wheelbarrows for delivery to the forms.

TYPE OF VIADUCT DESIGN

The viaducts are of reinforced concrete, with bents supporting a slab and beam deck. Together they form an X-shaped structure, with a solid fill at the intersection (between the two railways). The approaches and lateral connections are solid fills with concrete retaining walls. Each of the four sections of the viaduct has five spans, alternately long and short, with ¾-in. expansion joints over two of the bents (Fig. 5). At these joints the ends of the slabs have zinc plates which rest upon other zinc bearing plates on the cap girder of the bent. Concrete trolley poles are used.

The bents have square columns and footings spaced 12 ft. c. to c., with a guard wall extending between the

columns to a height of about 5 ft. above the rails, to protect the structure in the event of a train being derailed beneath it. The columns are capped with a heavy girder, on the faces of which are formed skewbacks for the floor-beams. In the Tower Grove Ave. viaduct, which is at right angles to the tracks, the floor-beams are

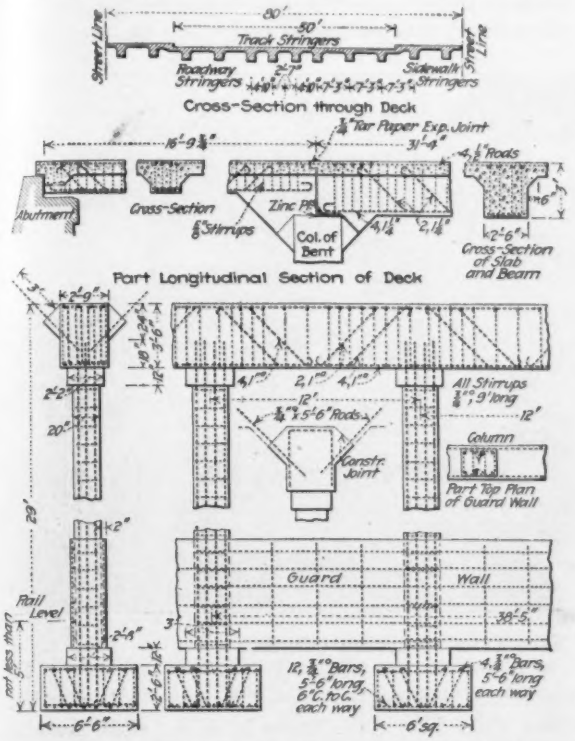


FIG. 5. THE TOWER GROVE AVE. VIADUCT

parallel to the line of the viaduct and are spaced to give two beams beneath each street-car track, as shown in Fig. 5.

On the Vandeventer Ave. viaduct, however, which makes a skew crossing with the tracks, the floor-beams are diagonal to the deck, as shown in Fig. 6. In this case, the ends of the beams and deck slabs are carried by deep facia girders, which are not required in the Tower Grove Ave. viaduct. It will be seen that for the longer skew spans (46 to 55 ft. long on the skew) steel facia girders are used, but these are incased in concrete so as to harmonize with the remainder of the structure.

HANDLING STREET TRAFFIC ACROSS THE WORK

One of the difficulties in construction was the necessity of keeping the streets open for pedestrians, street cars and vehicles. It was proposed at first to stop the cars at either side of the crossing, and let passengers walk across, placing temporary footbridges over the track-depression cuts. This would have been dangerous on account of the track crossings and would have caused much confusion owing to the different lines of cars that run on these two streets.

The car tracks were maintained therefore, and the temporary bridges over the cuts (which were built by the steam-railway companies) were made large and strong enough to carry the street cars. The bridges had to be shifted three times as the work progressed, so that very sharp curves had to be fitted for the track connections. Extra watchmen and policemen were provided to control traffic movements over the crossing, as long as the steam cars, street cars and other vehicles were operated on the same level. During the construction of the south end of the Vandeventer Ave. viaduct, street cars were diverted to the Tower Grove Ave. surface line by a temporary connection through property owned by the street-railway company. This is shown at *BB*, Fig. 1.

Later on, with the north end of Tower Grove Ave. viaduct built and the south end of Vandeventer Ave. also built, a timber bridge was built somewhat higher than the viaduct level, so that all street cars and vehicles could be carried above the work. This eliminated the grade crossing entirely, but introduced steep grades for the temporary approaches. The street cars were then run on a temporary track laid on the paving along Hunt Ave. and the viaduct approach (*CC*, Fig. 1) to a single track over this temporary bridge, and then diverted again through the connection noted above, but with the curves reversed (*DD*, Fig. 1).

For controlling traffic over the single-track temporary bridge, a pole at each end of this track carried a box containing two groups of incandescent electric lamps. Half the lamps were red and the others white. These were controlled by a man in a cabin at the center of the

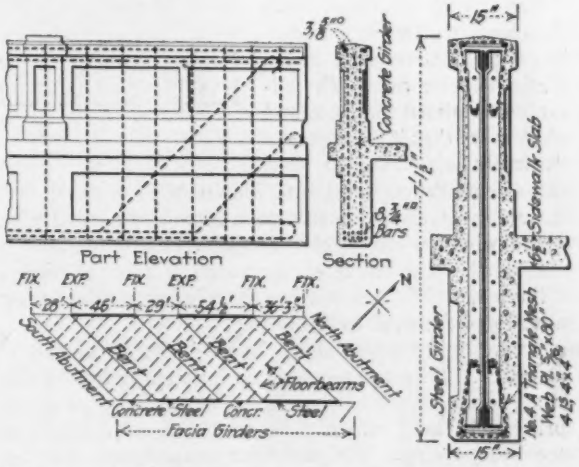


FIG. 6. THE VANDEVENTER AVE. VIADUCT (WITH FACIA GIRDERS SUPPORTING DIAGONAL FLOOR-BEAMS)

bridge, the movement of a handle causing the red lamps to be lighted at one end and the white lamps at the opposite end of the bridge.

United States-Canada Boundary Survey—Since 1908 surveyors have been at work marking the boundaries between the United States and Canada. The land surveys are nearing completion and the water lines of international lakes and rivers have been established. Charts of the international boundary, where the surveyors of the two countries have laid it down on maps of the water frontier, were on June 11 presented to Premier Borden by J. W. Stewart, head of the Canadian Hydrographic Survey. The Canadian Prime Minister, in receiving the charts of the boundary line from St. Regis Island in the St. Lawrence through Lakes Ontario, Erie, Huron and Superior to the head of the Lake of the Woods, referred to the happy relations existing between Canada and the United States as follows:

The fact that two great nations whose interests are concerned have chosen this method of bringing to a happy conclusion every possible difference is not only an example but a happy augury of future good relations. I trust that the work of delimiting the land boundary will be equally successful, and when this is done there will be set at rest every possible cause for dispute or controversy over the boundaries between the two countries.

A Novel Type of Motor Omnibus

The accompanying view shows an interesting design of motor omnibus which has just been completed by the McKeen Motor Car Co., of Omaha, Neb., for the Interurban Motor Transportation Co. operating between Minneapolis and St. Paul. The various details of the car body have been designed along lines similar to those used in the McKeen motor cars for railway service. The new vehicle is called the McKeen highway coach.

The chassis of the vehicle is the same as that used on a 3½-ton gasoline motor truck, and the propelling machinery is substantially identical. The wheel base is 18 ft. and the length of the frame back of the driver's seat is 22 ft. The passenger compartment is 18 ft. 2 in. long inside and 7 ft. 6 in. wide.

The round windows, with aluminum frames, are similar in design to those used on the McKeen motor cars. The advantages claimed for the round window are that it can be made absolutely air-tight, so that dust is entirely excluded, and the frame is so made that the rattling and disagreeable noise often experienced with square windows on street cars and motor omnibuses is eliminated. The frame of the round window is hinged at the top, and in hot weather the window can be swung inward and fastened to the ceiling.

A novelty in the seating of the car is the use of revolving chairs. These are mounted on telescoping iron standards, the weight of the chair being supported by a spiral spring 2 ft. long, which gives an elastic adjustment to secure easy riding. The passenger compartment contains 27 of these revolving chairs.

From the point of view of operating economy the most important feature in design is that the car is arranged to permit operation without a conductor, thus cutting in half the "platform-expense" item in operating costs. Passengers enter at the front through folding doors past a prepayment-fare collection box under the driver's supervision. The opening of entering and exit doors is controlled by the driver. The car draws up alongside the curb to take passengers on or let them off; the height of the step from the curb is only about 7 in.

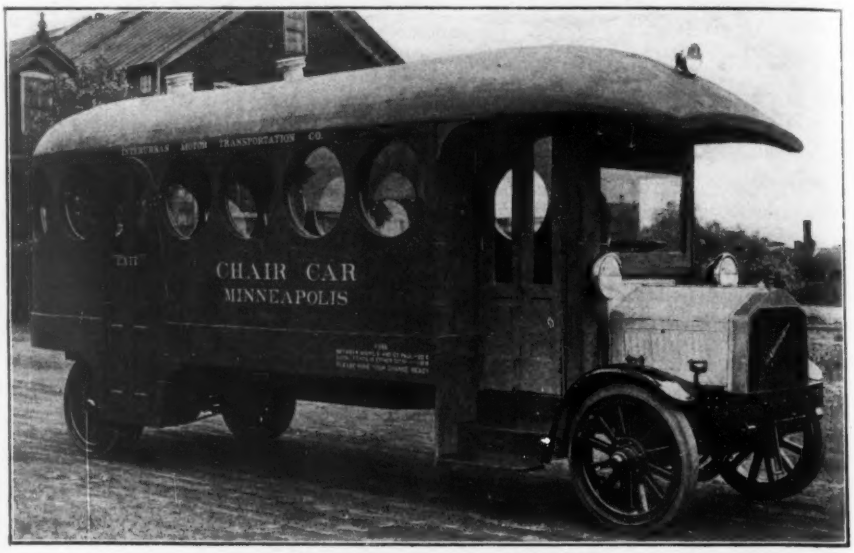
The coach is lit by electricity from the storage battery used for starting the engine and is heated by hot-water circulation from the engine jackets. The price of the coach at Omaha is $6500. The McKeen company plans to build a vehicle with a similar body and seating capacity for operation on railway track at a price of about $8000.

Improvements on the Chicago Great Western Ry. made during the last four or five years represent an average of $11,000 per mile for the entire system. This is as great an outlay per mile as was expended 30 years ago in the original construction of the railways built in the prairie states.

Three Residential Sewage-Treatment Plants Near Cleveland

By R. F. MacDowell*

Even with the smallest class of sewage-treatment plants, namely, those for residences and estates, there is often required much originality of design, so that plants of the same size, if properly designed to suit the local conditions, often differ considerably in their general features, especially with regard to the secondary treatment employed. This fact is brought out forcibly by a comparison of three small plants constructed near Cleveland during the past year. These plants were designed in the office of R. Winthrop Pratt and were constructed under his general supervision for three private estates, the owners of which, for convenience, will be called A, B and C.



A New Type of Motor Omnibus

A—Two-Story Sedimentation Tank and Glass-Covered Rapid Sand Filters

The estate and summer residence of Mr. A fronts on Lake Erie, and for many years the sewage has been discharged into the lake directly in front of his grounds. During 1914 Mr. A spent a large sum of money to improve his private bathing beach, which is also in front of his grounds, and he accordingly desired to provide other means of disposal of the sewage and kitchen wastes from his residence. It was therefore decided to construct a sewage-treatment plant on the grounds, having an outlet into a small stream which separates an adjoining estate. The plant was constructed at an inconspicuous location in the chicken yard in the rear of the garage. As the ground at this point was at too high an elevation to permit the construction of a gravity system, the sewage was raised to the plant by means of an ejector located in a manhole placed on the line of the existing sewer leading to the lake. The motor-driven compressor and air-receiving tank to operate the ejector were placed in the garage, which is 300 ft. from the ejector manhole.

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The sewage-treatment plant, which provides for the sewage of 30 people at the rate of 50 gal. per day, consists of a two-story sedimentation tank and two rapid sand filters, the latter being covered with glass. The filters were constructed adjacent to the tank, making a very compact layout which lies entirely within an area of 20x20 ft. As both tank and filters are built entirely in excavation, with covering, the plant is inconspicuous. The velocity of the sewage as it leaves the force main from the ejector is broken by discharging into a manhole 30 ft. from the sedimentation tank. The latter tank is 4½x8 ft. in plan and 8 ft. deep and has 2-in. wood flow-compartment walls and a 3-in. flat wood cover. A wrought-iron basket screen, which is placed inside the tank just below the inlet, may be removed by lifting a hinged door in the cover. As the plant will be used for only a short time each year, frequent removal of the sludge will be unnecessary, and hence no sludge-removal pipe was provided in the tank.

The filters, each about 6x11 ft. in plan, with concrete walls and floors, contain 4 ft. of filtering material, consisting, from bottom to top, of coarse slag, fine slag, pea-size silica gravel and lake sand. Only 3 in. of sand was used on the surface, and the sewage is thus filtered at a rapid rate. The sand was used merely for the purpose of distributing the tank effluent uniformly over the surface of the filtering material.

The degree of purification produced by the plant was not expected to be very high, but in view of the large dilution afforded the effluent by the stream it was considered adequate to eliminate all possibility of a "sight" nuisance being created. Furthermore, because of the coarseness of a large proportion of the filtering material, the intermittent discharge onto the filters by means of a dosing tank and siphon was considered unnecessary, and the latter were eliminated. After operating the plant for some time, however, it was found that even with only 3 in. of sand on the surface the filter clogged readily and became ponded, due principally, however, to a large quantity of grease from kitchen wastes. To remedy this, additional baffles were placed in the sedimentation tank and a glass covering was put over the filters. The purpose of the glass was to produce heat which would aid in drying the surface of the beds and thus make the scum and grease deposits more readily removable. The glass covering consists of an inclined, low, hip roof framework, holding hinged sashes in sections, the glass for which was lapped to shed water, as in greenhouse construction. Ventilators were placed in the top.

It is planned to use the plant during the bathing season only, while during the remainder of the summer the sewage will be discharged into the lake by means of a by-pass at the ejector manhole, the outlet being submerged in 5 ft. of water 100 ft. from the shore.

The total cost of the plant, including ejector equipment, was about $2000.

B—Screens, Two-Story Sedimentation Tank, Dosing Tank and Intermittent Sand Filters

The estate of Mr. B, for whom the second plant was built, is located on a ridge about 2 mi. from the lake. A very small stream, which practically dries up in the summer time, affords no dilution for the effluent from the plant. The grounds are of sufficient area and have suitable topography to suggest the construction of a subsoil irrigation, or absorption, system for the secondary treatment, but as the soil is a heavy clay and the sewage flow is comparatively large, it was considered more practicable to provide intermittent sand filters, especially since a location for open filters could be obtained which was well removed from dwelling houses.

The plant was designed to treat a maximum daily sewage flow of 4500 gal., based on 75 gal. per capita per day for 60 persons. The plant will be used throughout the year. The design provided for a screen chamber, a two-story sedimentation tank, a dosing tank, and two intermittent sand filters. The tanks are built together as one unit, and the filters are located 200 ft. away.

The sedimentation tank, which is built of reinforced concrete, with wood-plank covering and flow-compartment walls, is 6x10 ft. in plan and 10 ft. deep to the flow line. By means of a 6-in. cast-iron pipe leading to the bottom of the tank the sludge may be removed by gravity onto the surrounding corn field, where it may be plowed into the loamy top soil. A 1-in. water main leading to the bottom of the tank is also provided to assist in loosening the sludge. The dosing tank, which is 12x12 ft. in plan, is also covered with wood. A single 4-in. Miller siphon discharges the sewage onto the filters. The latter, which are 50x40 ft. in plan, or 0.046 acre in area, are constructed partly in excavation, with concrete walls. The filtering material, which has an average depth of 3 ft., consists of 6 in. of 1- to 1½-in. slag, on the bottom, and 2.5 ft. of river sand. The siphons discharge into a gate chamber at one end of the dividing wall between the filters. The flow onto either filter is controlled by hand-operated, metal slide gates. The sewage is distributed by means of wooden troughs, extending diagonally across each filter, each trough having hinged openings at intervals for distributing the sewage uniformly over the surface.

To keep out sticks, leaves, etc., during high water, the main underdrain from the filters is provided with a flap valve. The cost of this plant, exclusive of sewers, was about $1200.

C—Septic Tank, Dosing Tank and Sub-Surface Irrigation

The third plant, built for the country home of Mr. C, provides for only 10 to 12 persons, and is much simpler in design than the two other plants. There is no watercourse nearby, and as the soil is a good sandy loam, a subsoil irrigation, or absorption system, was designed for the secondary treatment. The plant was built on a steep hillside, and consists of a septic tank, a dosing tank and 600 ft. of 4-in. vitrified pipe, laid with open joints, in two lines, each at different depths from the surface, but at the same elevation, along parallel contours of the hill, 8 ft. apart. A single 3-in. Miller siphon gives intermittent discharge into the absorption system, and by means of a small concrete gate chamber one-half of the system may be cut out at any time. The vitrified absorption pipe is surrounded in the trench with 4 in. of screened river gravel and each joint is covered with tarred felt to keep out fine material. The tanks are oval-shaped in plan and are built of brick, plastered both outside and inside, and have concrete bottoms. The cost of the plant, exclusive of sewers, was about $350.

All three plants have operated successfully and have produced effluents of satisfactory character.

Hydrostatic Catenary Flume on a Concrete Aqueduct

By H. B. Muckleston*

SYNOPSIS—The Brooks Aqueduct, a concrete flume shaped to the hydrostatic catenary, a curve of zero moment and zero shear under hydrostatic load, supported on and a part of a reinforced-concrete trestle 2 mi. long. A part of the Canadian Pacific Railway's Alberta irrigation scheme. Carries on hydraulic gradient a maximum flow of 900 cu.ft. per sec. Has reinforced-concrete siphon under railway tracks.

The Brooks Aqueduct plays a very important part in the system which has been built to serve the Eastern

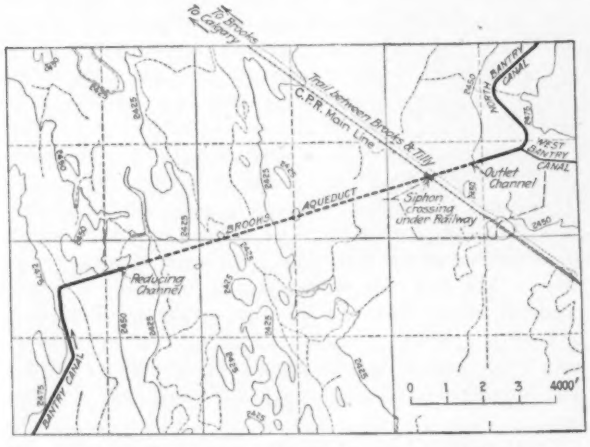

Fig. 1. Location of Brooks Aqueduct in Southern Alberta Irrigation System of Canadian Pacific Ry.

section of the Canadian Pacific Railway Company's irrigation block in southern Alberta. The system is supplied with water from an intake in the Bow River near the town of Bassano.† The main diverting canal cuts through the watershed between the rivers and reaches bench elevation after a course of about five miles. From this point it runs in a southeasterly direction, and after supplying many branches spills its surplus waters into Lake Newell. This lake is a storage reservoir with a useful capacity of 187,300 acre-feet. It occupies a depression to the west of a low range of hills and is designed to be filled during the nonirrigating season. From the reservoir the aqueduct is supplied by a short canal.

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†The headworks were described in "Engineering News," Aug. 27, 1914, p. 425, and Sept. 3, 1914, p. 484. See map of system given there.

A marked feature in the major topography of the project is the existence of several deep valleys, which cut clear across from one river to the other. One of these is utilized by the main canal; another is crossed by the East Branch at Antelope Creek, and a third is situated just east of the reservoir and is crossed by the Brooks Aqueduct. The aqueduct is required to carry water for the irrigation of 125,000 acres of land, and its discharge capacity is required to be 900 cu.ft. per sec.

Controlling Factors of Design

The reservoir is a very important factor, not only in the design of the canal system as a whole, but in the design of the aqueduct. By the Canadian law the duty of water is fixed at 1 cu.ft. per sec., delivered at the farm boundary, for each 150 acres of irrigable land flowing continuously for 153 days, or, in round figures, about two acre-feet per acre during the season. To this must be added about 25% for losses in transit through seepage and evaporation, thus requiring the diversion of about 2.5 acre-feet for each acre of irrigable land. The legal season is from May 1 to Oct. 1, and during the remainder of the year, under usual conditions, the system would be empty. However, if facilities can be provided in the lower portion of the system for storing up the otherwise useless winter flow of the river for use during the following summer, the discharge capacity of the upper part of the system can be correspondingly reduced. Such storage, therefore, has a definite and easily ascertainable value, which fixes the maximum allowable cost of developing it. It is evident, too, that any works which tend to reduce this reservoir capacity must have a value in some other direction in excess of that of the lost storage, or they would not be economical.

Now, the elevations of high water in the reservoir and of the discharge end of the aqueduct were fixed by

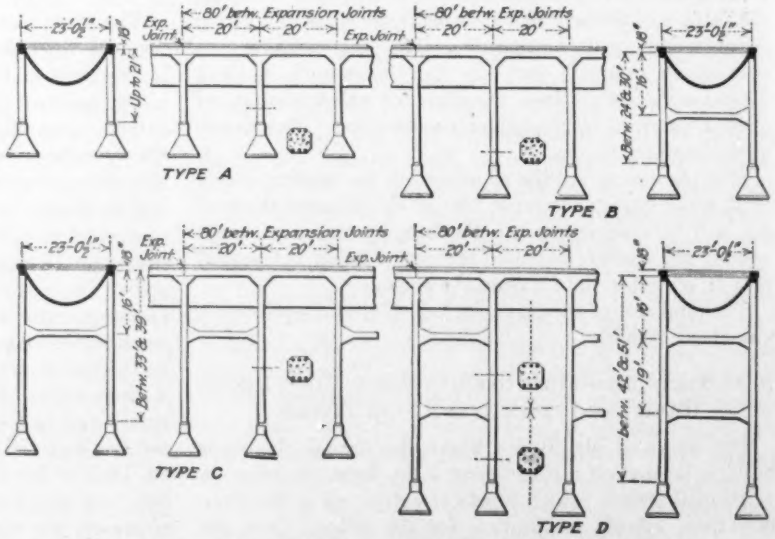


Fig. 2. General Design of the Brooks Aqueduct Showing Variation in Type of Structure

other considerations and could not be altered. That is to say, the total available fall between high water in the reservoir and the outlet end of the aqueduct was a fixed quantity, of which a certain portion would be required for the slope of the outlet canal and aqueduct and the remainder available as storage. Consequently, the depth and quantity of available storage in the reservoir depended directly on the slope given to the aqueduct. This latter might be chosen at any value between somewhat wide limits, and the cost of the structure would vary inversely as some function of the value chosen.

The cost of the head lost in the aqueduct may be figured in two ways: First, it represents a certain direct expenditure in the cost of the aqueduct, varying inversely with the slope given to it; second, it represents a certain indirect expenditure, equal to the value of the lost storage, varying in some degree directly with the slope; and there will be some value where the sum of these two is a minimum. In order to determine this point, it was necessary to know, first, the value of the lost storage per acre-foot; second, the probable cost of the aqueduct over a considerable range in slope.

The value of the lost storage is very nearly a constant quantity, and is equal to the cost of the canal system supplying this reservoir divided by its average annual discharge in acre-feet. The probable cost of the aqueduct involves questions of type of structure and material to be used, and can only be solved by a process of cut and try on a number of examples.

For a preliminary estimate it was assumed, first, that a pressure pipe or inverted siphon would be the cheapest type; second, that the cost would be a direct straight-line function of the cross-section area. Approximate designs were worked up for steel, concrete, and wood-stave pipes and estimates made, allowing for maintenance charges in each case. From these estimates the probable extreme values of the lost storage were deduced.

While in many cases, especially where the depth of the valley is great, a siphon is cheaper than a structure on the hydraulic grade with free surface, it is not always so; and even when it is cheaper, siphons as the solution of such problems are open to many objections from which flumes are free. In the first place, canals do not always run full, whereas the siphon must always do so. This condition has two very disagreeable effects. The low velocity in the siphon causes the hydraulic grade to flatten out, resulting in a drop at the upper end, which, in a large canal at any rate, requires control works; otherwise the effect of the fall will be felt for a long distance above. Another effect is the accumulation in periods of little demand for water of sediment in the bottom of the pipe, which is caused by the low velocity at small discharges. The only cure for these effects is to divide the full discharge among several pipes and provide means of cutting out one or more pipes as the discharge decreases. This remedy costs money and, if the available head is limited, is not always feasible. Another objection is the necessity in a northern climate of blowing off every fall.

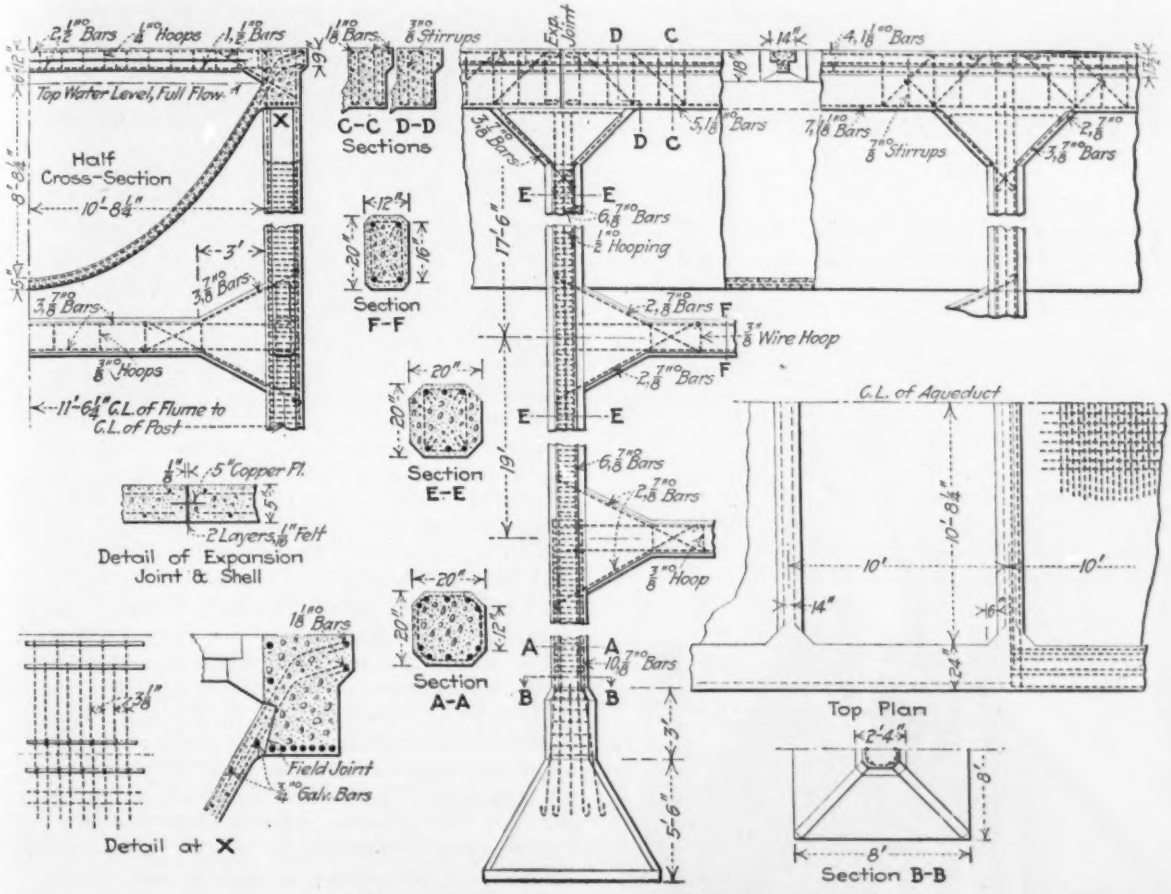


FIG. 3. TYPICAL DETAILS OF BROOKS AQUEDUCT

In a large pipe under high head, especially if very long, this must be done very slowly; otherwise serious pressure oscillations are liable to be set up, giving rise to stresses which are difficult, if not impossible, to estimate with any accuracy.

For these reasons and some others it was decided to investigate the possibilities of a grade structure. Accordingly, a tentative design was worked out and its probable cost compared with that of a pressure pipe, using the same hydraulic slope in each case. The cost of the grade structure proved to be somewhat less than that of a single pipe and very much less than that of a double one. Consequently, subsequent investigations were confined to the grade structure. Four values of lost head were then assumed, two on either side of the probable final value, and four approximate designs were made, from which a cost curve was plotted. From this curve and from a similar curve for value of lost storage the final value of the lost head was determined.

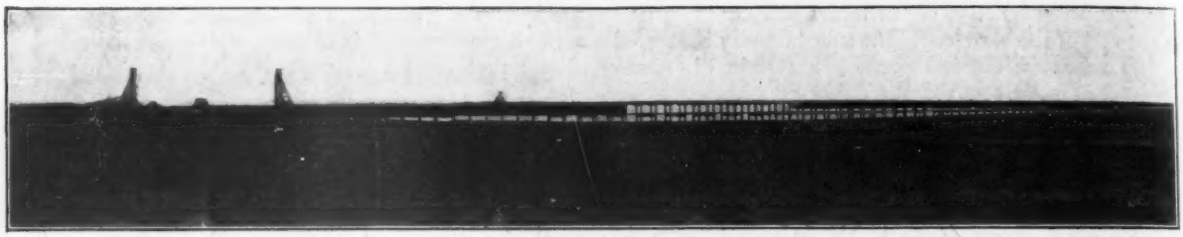


FIG. 4. VIEW OF ONE END OF BROOKS AQUEDUCT IN SOUTHERN ALBERTA

FIG. 5. ERECTING THE BROOKS AQUEDUCT

DESIGN OF AQUEDUCT

The final design (Fig. 2) consists of a curved shell of reinforced concrete, 10,480 ft. long, suspended between horizontal girders, the whole being supported by a reinforced-concrete trestle made up of two lines of columns spaced 20 ft. c. to c. and varying in bracing according to the height, which increases from 24 to 60 ft. The novelty in the design is in the curve adopted for the suspended shell. The suspended type in itself is not new, being familiar in steel construction. In such cases, however, the curve used is almost invariably circular, probably from reasons connected with manufacture. In the case under discussion the curve chosen is the hydrostatic chord, or elastic curve, sometimes called the hydrostatic catenary. It is the curve which the shell would assume were it made of some perfectly flexible and inextensible material. It is also one of the forms assumed by a straight spring when the ends are acted on by two equal opposite forces in the same line. The radius of curvature at any point varies inversely as the distance from the water surface. Consequently the shell tension is uniform throughout, and there is neither moment nor shear at any point.

It is evident that the initial angle may be anything between very wide limits, and hence it must be chosen to satisfy external considerations. In this case hydraulic considerations pointed to an angle somewhere about 60°; and as this angle also happens to be suitable from the structural point of view, it was chosen as the value to be used. The best value for hydraulic properties only may be most easily determined by a little experimenting with a piece of clock-spring and a planimeter.

Computation of Curve—In order to calculate coördinates to the curve for the purpose of plotting and to determine the area and wetted perimeter, recourse is had to the fact that the curve is the equilibrium polygon for the liquid load filling it to the bowstring. A rigid analysis by calculus leads to a differential equation which the writer was unable to integrate. However, the equilibrium polygon furnishes an approximate solution, which gives results close enough for practical purposes.

Since the load is normal to the curve and the tension uniform, it follows that the load line in the force diagram must be the arc of a circle, the radii of which represent the tension in the chord. With the initial angle given and the required area known it is a simple matter to draw the force diagram.*

*See a method of computing the hydrostatic catenary, by H. M. Gibb, the detailer of the Brooks Aqueduct, in "Engineering News," Apr. 8, 1915, p. 668.

The curve used for the Brooks Aqueduct is figured out in the table at the bottom of the page, in which (*a*) is the force diagram and (*b*) is the beginning of the catenary as drawn from that diagram.

Stress Computations—The thickness of the shell is determined by two requirements—(1) when the channel is running less than full there exist both moment and shear, and the shell must be designed stiff enough to resist these without cracking; (2) when running full, the concrete shell must not crack in tension. To meet these requirements, a sufficient area of steel was provided to take all the tension at 10,650 lb. per sq.in. The elastic ratio was assumed at 15, and a sufficient thickness of concrete was then added so that, assuming the concrete and steel to share the tension, the stress in the concrete should not exceed 70 lb. per sq.in. The steel was then distributed in two layers, as far apart as the thickness so found would allow, and the shell investigated for deflection, moment and shear under partial loads. The stresses thus calculated were well within the specified limits and the deflections very moderate.

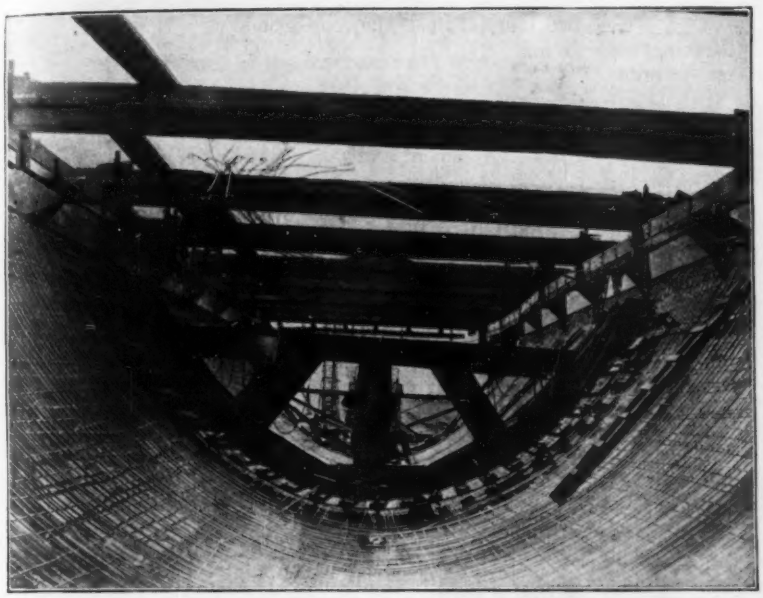


FIG. 6. CONCRETING THE FLUME

Calculations of the Discharge—The discharge calculations were made by an exponential formula instead of by the usual Kutter formula. The latter formula, while very widely used, holds its place in hydraulics largely because of the very extensive collection of tables and diagrams which have accumulated around it. The exponential formulas require no such aid, are as easily handled and are believed more closely to fit observed results than the older and much more complicated Kutter formula. The particular formula used is attributed to Prof. T. Claxton Fidler, of St. Andrews University, Dundee, and is as follows:

$$Log\ S = X\ (M + log\ V - \tfrac{2}{3}\ log\ R) - 6.00$$

S, *V* and *R* have the usual significance, *X* is a friction factor depending on surface and *M* a friction factor depending on shape. For any given relations between area and hydraulic radius the formula is readily transposed to give friction head direct for any required conditions. It is thus simpler than Kutter's formula for channels of fixed shape, such as pipes, flumes and the like.

CALCULATION OF HYDROSTATIC CHORD FOR BROOKS AQUEDUCT

Draw AB equal, by a convenient scale, to the desired area of cross section, and draw AC and BC making the angles with the horizontal at A and B equal to the required initial angle (in this case 60°). Draw the arc ADB. Divide the arc into any convenient number 2r (in this case 20) of equal parts, the more the better, and compute the length of one part by the scale. Call this L; it represents the equal areas into which the catenary section is divided. That is, in the diagram (b), $hx = L$, where h is the head on any one of the sections and x is the chord of the curve being sought. The table herewith is then worked out, giving the ordinates of the curve from point to point. This table merely gives in tabular form the steps in the derivation of the formula in Col. IX, which is a geometric solution of the chord length, from which the ordinates and abscissæ (Cols. XII and XIV) are readily deduced. The geometric solution of L (see b in table) is as follows, solving in this case for x_1 and x_2:

$h_1 x_1 = L$

$h_1 = \frac{x_1}{2} \sin \theta_1$

$\therefore L = \frac{x_1^2}{2} \sin \theta_1$

$2L = x_1^2 \sin \theta_1$

Further:

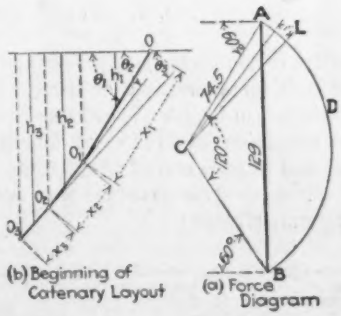
$h_2 x_2 = L$

but from diagram (b)

$h_2 = (\frac{x_2}{2} + \frac{x_1 \sin \theta_2}{\sin \theta_1}) \sin \theta_2$

$2L = x_2^2 \sin \theta_2 + 2\ x_1 x_2 \sin \theta_1$

which is the formula in Col. IX.



Twice the sum of Col. X gives the perimeter, nearly. As a check on the work, add the ordinates in pairs and multiply by the figures in Col. XIII; the sum of the results (Col. XV) should be slightly less than the required area. To insure accurate results, it is advisable to work to several places of decimals.

$a = 6°0'$, $r = 10$. $L = 7.825$. $A = 129$.

I	II	III	IV	V	VI	VII	VIII	IX	X		XI	XII	XIII	XIV	XV
n	(r−n+½) a	Sin. II	(r−n+1½)a	Sin. IV	2n−1	V×VI	Prog. Sum. VII	III x^3n+VIII x_n=2 L	x_n	Point	X×III	Ordinate	X×Cos II	Abscissa	A
1	57°	0.839	63°	0.891	0	0	0	$0.839x^3$	4.32	0	....	0	....	0	.....
2	51°	0.777	57°	0.839	8.64	7.24	7.24	$0.777x^3 + 7.24x$	1.89	1	3.62	3.62	2.36	2.36	8.55
3	45°	0.707	51°	0.777	3.79	2.93	10.17	$0.707x^3 + 10.17x$	1.40	2	1.47	5.09	1.19	3.55	10.36
4	39°	0.629	45°	0.707	2.80	1.98	12.15	$0.629x^3 + 12.15x$	1.22	3	0.99	6.08	0.99	4.54	11.05
5	33°	0.545	39°	0.629	2.43	1.53	13.68	$0.545x^3 + 13.68x$	1.10	4	0.77	6.85	0.95	5.49	12.30
6	27°	0.454	33°	0.545	2.20	1.20	14.88	$0.454x^3 + 14.88x$	1.00	5	0.60	7.45	0.92	6.41	13.15
7	21°	0.358	27°	0.454	2.00	0.91	15.79	$0.358x^3 + 15.79x$	0.95	6	0.45	7.90	0.89	7.30	13.67
8	15°	0.259	21°	0.358	1.90	0.68	16.47	$0.259x^3 + 16.47x$	0.90	7	0.34	8.24	0.88	8.18	14.20
9	9°	0.156	15°	0.259	1.80	0.47	16.94	$0.156x^3 + 16.94x$	0.85	8	0.28	8.52	0.87	9.05	14.58
10	3°	0.052	9°	0.156	1.70	0.27	17.21	$0.052x^3 + 17.21x$	0.80	9	0.13	8.65	0.84	9.89	14.42
									14.43	10	0.04	8.69	0.80	10.09	13.86
								P = 28.86							126.14
								Hydr. rad. = 4.48							

Siphon—The site of the aqueduct is crossed by the main line of the Canadian Pacific Ry. This track is at such an elevation that sufficient headroom for an overcrossing could not be obtained and a siphon under the track was necessary. In order to save excavation, and more especially for reasons of drainage, the siphon was designed with a reduced section and an increased velocity on the principle of the Venturi tube. The general layout of the siphon is given in Fig. 8.

Trestle Design—In designing the trestle carrying the flume a good deal of preliminary work was done in order to arrive at the most economical type. The final design is shown in Figs. 3 and 4. In order to keep down form costs it was desired to have as little variety in spans as possible, even if quantities were somewhat increased thereby.

The trestle is divided into four typical constructions, depending on the height of the columns. When the height of the columns changes, the change is invariably by a step of 3 ft. or a multiple of three. Provision is made for variation in length of shell due to change of temperature by expansion joints at every fourth bent and at each portal and at each end of the siphon.

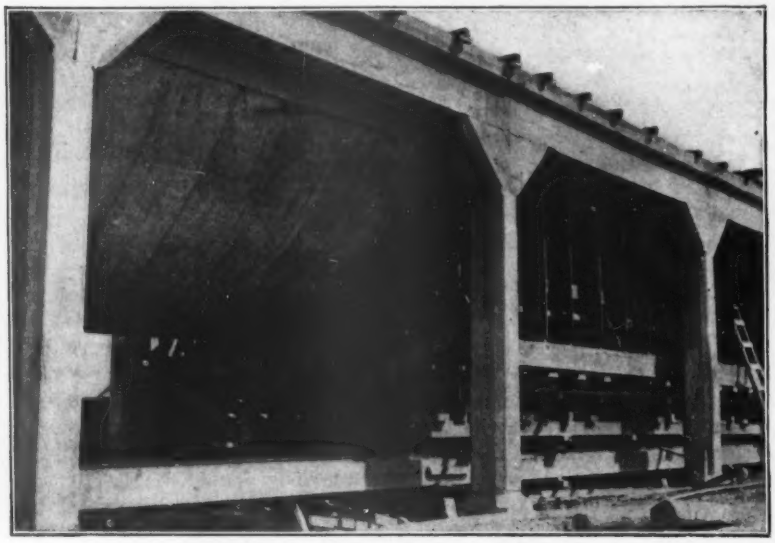


FIG. 7. NEAR VIEW OF OUTSIDE OF FLUME

Stresses—The structure was designed for the following unit stresses:

	Lb. per Sq.In.
Concrete-bearing stress	400
Direct compression—massive work	400
Compression in flexure, reinforced	750
Short columns, not over 18 in. diameters. Not hooped	400
Hooped columns	600
Shear in flexure	60
Bond plain bars	60
Bond deformed bars	120
Steel tension	16,000

The structure was designed to resist wind pressure of 30 lb. per sq.ft. in any direction on all exposed surfaces. The range of atmospheric temperature was assumed from —45° to +85°, and full allowance was made in the design for all stresses caused thereby.

Foundations—The foundations for the aqueduct are carried to a depth of at least 7 ft. below the original surface, so as to be sure of getting below any possibility of heaving by frost. At this depth the material as exposed in foundation pits is a stiff sandy clay, which might almost be classed as a weak sandstone. In its natural state or when freshly exposed it is quite firm, but it disintegrates rapidly on exposure to the atmosphere. Test loads were applied to surfaces varying from 36 to 324 sq.in. in area and the sinking under various loadings observed. Under 18 tons per sq.ft., on 36 sq.in., the settlement was about 0.20 ft. when dry and 0.31 ft. when wet. As the service load under maximum wind pressure is only about two tons per foot, the foundations were considered satisfactory.

CONSTRUCTION

Railway sidings were built near the crossing, storehouses for cement and other material erected and a concrete-lined reservoir for water was constructed. A double-track, standard-gage railway, with frequent crossovers, was built along the line of the work on the south side, and a pipe leading from the reservoir to an elevated tank at the west end of the structure laid down, with taps at frequent intervals. Column, girder and brace forms were erected by stiff-leg derricks traveling on skids and working backward as the forms were put up. Outside shell forms were mounted on a carriage which traveled on rails carried by the upper cross-struts, the forms being raised into place or lowered by screwjacks. A locomotive crane was used for various purposes and a Shay engine for switching cars and pulling the outside shell forms ahead.

The pedestals were poured by a portable mixing plant with a dwarf hoist and spout, from four to six pedestals being cast from one setting. As soon as the concrete had set, the forms were stripped and the pits backfilled. For the construction of everything except the pedestals, the concrete hoisting and mixing plant was mounted on flat cars, which traveled on the track nearest the structure. The gravel for concrete was all shipped in by rail in steel side-dump cars. A small reserve storage was kept, but most of the requirements were filled direct from the cars. A special chute with gate was constructed which could be fitted to the cars and which allowed them to discharge directly into the hopper of the mixer, the cars standing on the second track alongside the mixer car.

All reinforcing steel, except that for the shell, was shipped cut to length. All bending and the assembling of the column reinforcement were done in a central yard. The shell steel, which consisted of No. 0000 galvanized-steel wire, was shipped in coils and, after being passed through a straightening machine, was cut to length by a power shear. Steel for braces, girders and the shell was assembled in place and wired.

The first step after the pedestals had been stripped and backfilled was to erect the column forms, which were securely guyed and cross-braced. Forms for the braces, if any, and the girders were next set up and the reinforcement placed in them and wired. The girder cross-struts were precast in yards and hoisted and placed by the locomotive crane. The columns and braces were poured in a single lift, followed by the girders after an interval

to allow for setting and shrinkage. After the concrete had set sufficiently, the column, brace and girder forms were stripped and the track for the outside shell forms put in place (Fig. 7). The outside forms for the shell were then pulled ahead and jacked up to correct elevation, followed by the placing of the shell reinforcement, which was held in position by concrete spacing blocks grooved to hold the wires. Inside forms for the shell were handled from Π-tracks suspended from the cross-struts (Fig. 6), and were placed as soon as the steel had been put in.

The pouring of the shell was done in alternate 80-ft. sections. A good deal of difficulty had been anticipated in making a good job of casting this thin member, and the inside forms first provided had removable panels or doors so that the lower part could be poured from near the bottom. It was found, however, that if a sufficiently fine aggregate were used and the mixture kept very wet, little or no difficulty was experienced, and the doors were abandoned as unnecessary. The secret of success was found to lie in pouring batches alternately from each side, and in rapid work, thus insuring a constant backward and forward movement of the fluid concrete crossways of the shell until it was full. This method made certain that the reinforcement was well surrounded and that no voids were left unfilled. This was only possible because the running foot quantities were small and a whole section could be cast in a short time before the concrete first poured could take its initial set.

It was found in building the experimental section that the shrinkage in setting lifted the concrete off the outside forms. This had been expected, and the inside forms had been stripped as soon as the concrete was sufficiently hard to stand alone on the steep side slopes. The same thing was anticipated in building the aqueduct, and to prevent possible cracking from this cause, the inside forms were stripped within a few hours after pouring.

The construction of inlet, outlet and siphon, apart from rather complicated formwork, presented little that was in any way unusual. As they could be used but once, the forms were built up on the spot and wrecked when the concrete had set.

During construction of the aqueduct temporary bulkheads were built at several points and the shell filled with water for testing. The object was principally to test the efficiency of expansion joints, and no attempt was made to measure deflections. They must have been very small, however, as the leakage under the bulkheads was almost negligible.

Personnel

The general design of the structure and the method of calculating the curve were worked up by the author. The detailed design is the work of H. M. Gibb. A. S. Dawson was the Chief Engineer for the railway company. The contractors were Grant Smith Co. & McDonnell, Ltd., of Vancouver, R. V. Engstrom being their Superintendent of construction.

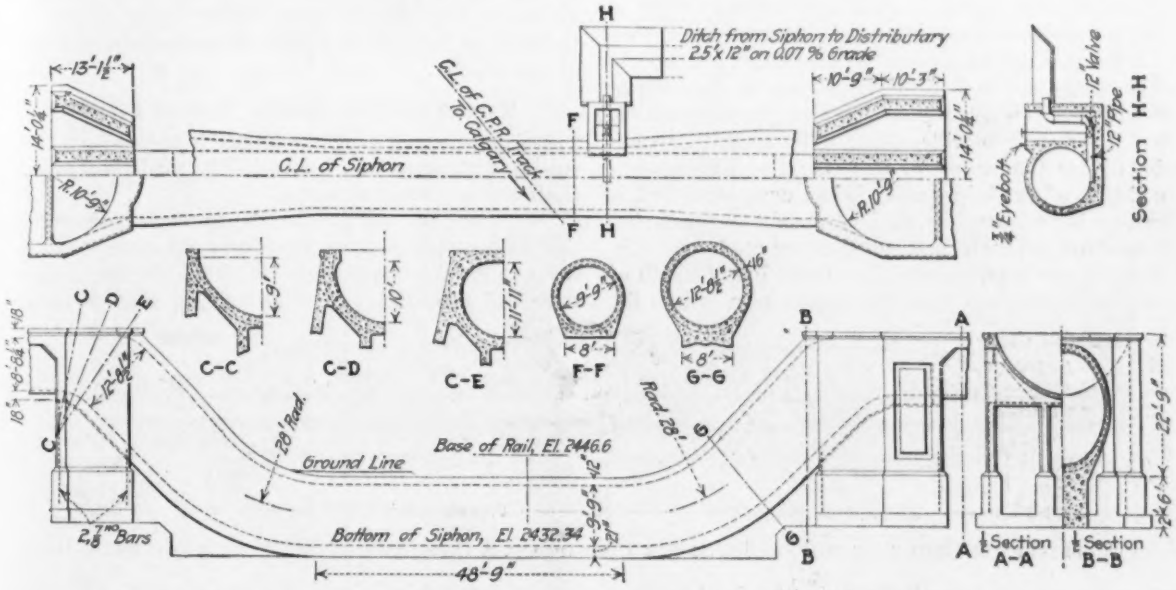


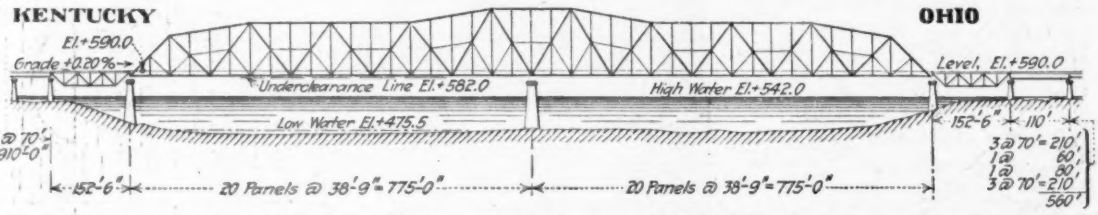
Fig. 8. Details of Siphon Carrying Brooks Aqueduct under Railway

Street-Railway Cars in Christiania—The excellent electric street-car service and the luxurious-looking street cars in Christiania, Norway, are mentioned by Commercial-Attache Henry D. Baker, of Petrograd, in a recent report to the United States Department of Commerce. The Christiania street cars are painted bright red, green, blue, etc., according to the lines on which they are operated. They are conspicuous for their large windows and clean curtains. The device so familiar to street-car patrons in the United States, namely, the strap, is absent, but on the backs of the seats (which face forward) there are small niches which afford passengers standing in the aisle convenient grips for the hand. On the platform are small metal receptacles into which passengers can throw used-up tickets, remains of cigars, etc., instead of dropping them onto the floor. Mr. Baker mentions an interesting advertising device he observed on these cars: Across the ceiling of the car, midway between the two ends, is a double-faced clock bearing the advertisement of a certain locally made shoe. A clock is always an object of interest and attention, so in this case the advertisement must perforce be read. On the outside of the cars are a number of hooks to which passengers can attach their skis. The fare within city limits is the equivalent of three cents; to outlying districts about eight cents. The conductor is paid $27.04 per month net after deducting payments for uniforms, sick funds and widows' pensions. The population of Christiania is about 250,000, but within a range of 10 mi. there are 750,000 people.

Long-Span Continuous-Truss Bridge over the Ohio

SYNOPSIS—Requirement of two 750-ft. openings led to adoption of two-span continuous truss; first large bridge of continuous type in America; piers on rock, eliminating the question of settlement; triangular web system with all joints riveted; enormous gussets required; floor-beams of U-shape, sides extending up the posts to the bottom of the swaybracing, which reduces the bending-moment in the floor-beam nearly one-half; traction truss in every panel; no expansion joints in floor

Striking departure from conventional practice in bridge design is found in the Chesapeake & Ohio Northern Ry.* bridge which is now under construction over the Ohio River at Sciotoville, Ohio. The consulting engineer, Gustav Lindenthal, of New York, who worked out the design of the bridge and viaduct approaches and is directing the construction, decided upon a two-span continuous-truss bridge as best suited to the conditions. The bridge has two equal spans of the extraordinary length of 775 ft. each, continuous over the center pier. Since the continuous truss is practically unknown in American bridge practice except for its use in swing bridges, the radical character of the Sciotoville design is evident.



NEW DOUBLE-TRACK BRIDGE OVER OHIO RIVER FOR CHESAPEAKE & OHIO NORTHERN RY. AT SCIOTOVILLE, OHIO

CONTINUOUS SPANS

The river is about 1500 ft. wide at the bridge site, and makes a sharp curve with the channel near the inner, or Kentucky, side. At high water, however, the river traffic shifts over to the Ohio side. These circumstances made it desirable to have two large openings, fixed by the War Department at 750 ft. This called for two spans of about 775 ft. between centers of piers. A comparison among various designs consisting of successive simple spans, cantilever trusses and continuous trusses showed a decisive economy for the adopted design, in addition to which it has the advantage of symmetrical appearance.

Foundation conditions for the river spans are excellent. Rock was found in the middle of the river at an elevation of about 10 ft. below low-water level, and it extends nearly horizontally across the river. Since the concrete piers will rest on solid rock, settlements are not to be expected, so that one of the common objections to the continuous type of bridge is not present in this case.

*The Chesapeake & Ohio Northern is a new branch line of the Chesapeake & Ohio Ry., to connect the main line of the latter on the Kentucky side of the Ohio River with the Hocking Valley R.R. at Columbus, Ohio. It crosses the Ohio at Sciotoville, about 120 mi. above Cincinnati. The new line is projected for heavy freight traffic, principally coal trains, for which reason the grade has been limited to 0.2%, compensated on curves.

The principal dimensions of the river spans are as follows:

Total length c. to c. end piers	1550	ft.
Span length c. to c. of piers	775	ft.
Clear height above low water	106½	ft.
Clear height above high water	40	ft.
Height of trusses at center pier	129	ft. 2 in.
Height of trusses at ends	77	ft. 6 in.
Width between centers of trusses	38	ft. 9 in.
Panel length	38	ft. 9 in.

The two large channel-spans are being built double-track. The approach viaducts, on the other hand—consisting largely of plate-girder spans 67½ to 110 ft. long—are being built for the present with steelwork for one track, while the masonry is built to accommodate a future second track.

TRIANGULAR WEB SYSTEM, RIVETED JOINTS

A Warren web system was applied to the adopted truss outline, with main-panel length 155 ft., subdivided by verticals and by a substrut system.

Most chord and web members being subject to both tension and compression, on account of the continuous design and the web system employed, it was decided to make them stiff throughout. It appeared also that no saving could be effected by the use of eye-bars for the few members taking tension only.

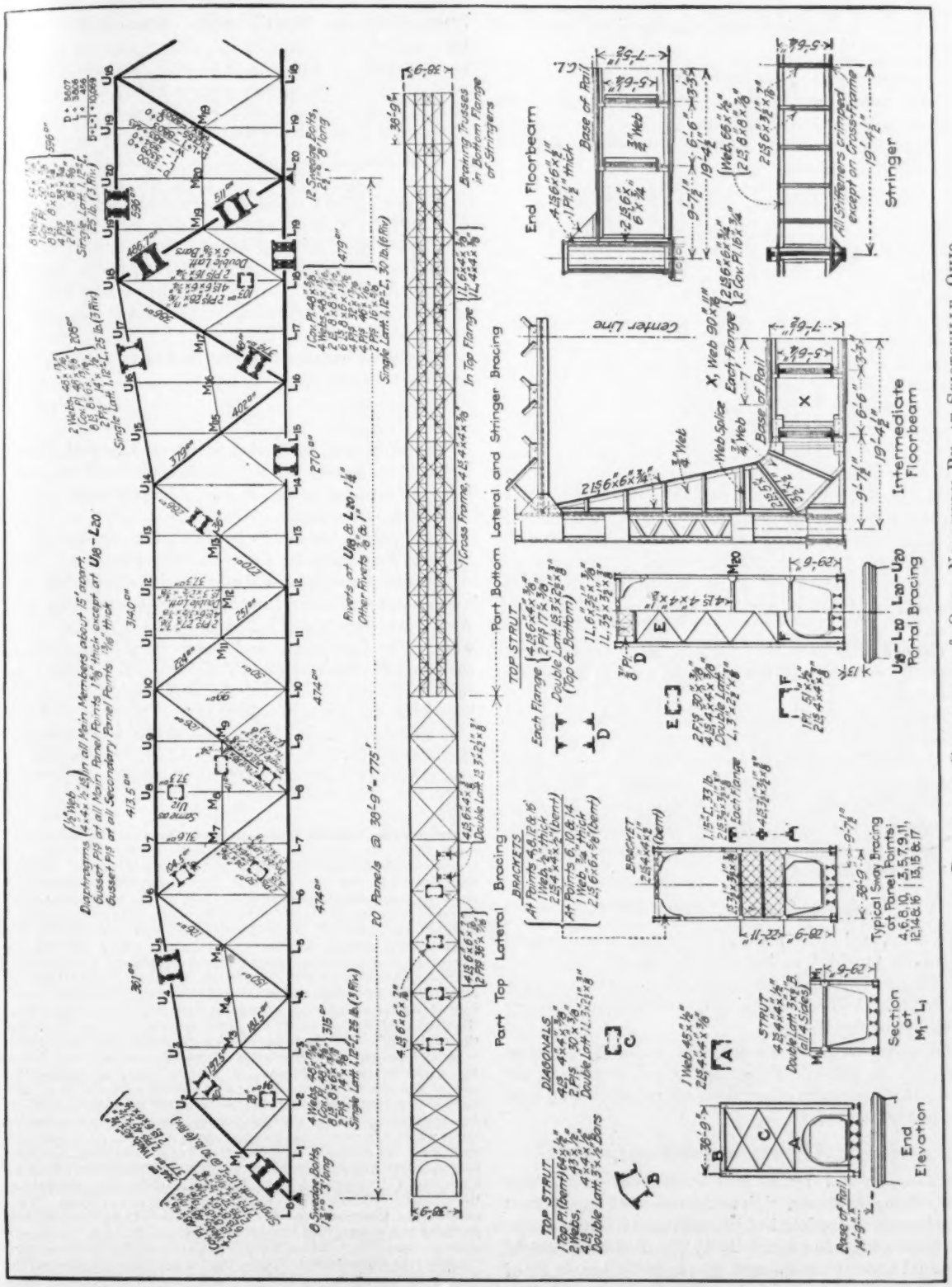
Pin connections being considered undesirable for members subject to reversal of stress, it was decided to use riveted connections throughout and thereby secure greatest possible rigidity. These spans will be the longest and heaviest of the fully riveted type built in America.

The riveted connections involve tremendous gusset-plates, larger, in fact, than those of the Hell Gate arch and of the same thickness—1⅝ in. The facilities of mills and bridge shops will be taxed to the utmost in providing these gussets.

Even so, it was not possible to provide gusset connections of the ordinary type at panel-points U18 and L20 (that is, at both ends of the main-pier diagonal). The stresses in the members that connect at these points are so great that it was found impracticable to make the connection to the gussets with rivets in double shear without exceeding the obtainable size of plates. Therefore, each web-plate of the top chord and diagonal members, as well as the gussets at U18 and L20, was made of four $\frac{13}{16}$-in. plates so arranged that the rivets can transmit the stress in shearing in four planes. The aggregate thickness of each gusset at these two points is 3¼ in.

Sizes and weights of individual parts in the bridge are given by the following tabulation:

Heaviest chord member, 4x4½x77½ ft.; cross-section, 596 sq.in.; weight, 228,000 lb.



STRESS SHEET AND DETAILS OF CHESAPEAKE & OHIO NORTHERN RY. AT SCIOTOVILLE, OHIO

EXCESS LOAD PROVISIONS IN SCIOTOVILLE BRIDGE

Wind load, top chord, 800 lb. per lin.ft. bridge.
Wind load, bottom chord, 700 lb. per lin.ft. bridge.
Wind load, train, 500 lb. per lin.ft. bridge.
Lateral force, 10% of live-load of one track.
Braking, 670 lb. per lin.ft. truss.
Excess = (W. + Br.) — 20% (D + L + I + Lat.)

Field connections of stringers to floor-beams not riveted up until span is completely erected and self-supporting. Field connection of diagonal M19 — L20 to bottom-chord gusset not reamed to full size or riveted up until span is self-supporting.

Heaviest web member, 4x4½x75 ft.; section, 511 sq.in.; weight, 166,000 lb.

Largest gusset-plates, 135x1⅝ in. by 14 ft. 9 in., and 140x1⅜ in. by 18 ft. 2 in.

Heaviest casting, 45,000 lb.

Largest rivets, 1¼-in. diameter by 7⅝-in. grip.

Each center bearing carries a dead-load of nearly 5000 tons and a live-load of 3000 tons.

All joints of members, whether in tension or compression, are fully spliced.

Floor-Beam Construction

The floor is no less remarkable than the trusses. The floor-beams are U-shaped frames, with sides extending up the posts to the bottom of the sway-bracing. This new form was used because of the shallow depth available; ordinary floor-beams would not have been as rigid and would have weighed considerably more. The computed center moments of the U-shaped floor-beam and of the end floor-beam—which is of ordinary construction—are 6,300,000 and 8,197,000 ft.-lb. respectively, and their flange areas are 42.63 and 55.17 sq.in.; but the end floor-beam carries only half a panel-load, and its moment and flange area must therefore be increased in ratio of the end shears (including impact) to compare the two types fairly. Making this correction, the plain floor-beam has a moment of 11,700,000 and a flange area of 78.8, against 6,300,000 and 42.6 for the U-shaped floor-beam. This reduction of bending moment is, of course, due to the fact that the upper ends of the U are held apart by the swaybrace strut.

The deflection as well as the moment of the floor-beam is greatly reduced, and the bridge as a whole is stiffened transversely.

Transverse stiffening and bracing effect is further secured by the type of sway-frames and portals employed. Solid plate-girder portals, forming an inverted U which extends down to floor level, are provided in the planes of the inclined posts at end and middle piers. For intermediate sway bracing, in place of the diagonal sway bracing ordinarily employed, deep lattice-girders and curved knee-brackets are used.

Traction Truss in Every Panel

In each panel in the plane of the bottom lateral bracing a truss is provided to transmit the longitudinal "traction" due to braking of trains directly to the trusses without bending the floor-beams. The arrangement is shown by sketch in the drawing.

No expansion joints are provided in the floor system. However, to prevent setting up chord tension in the stringers, the stringer connections are not riveted up until the spans are swung.

Design Loading

The design and details were worked out with a view to obtaining a strong, rigid structure that would meet the demands of the heaviest immediate and future traffic. The live-load is E60 on each track. The dead-load (weight of steel 13,500 tons) amounts to 18,800 lb. per lin.ft. of bridge. A lateral force equal to 10% of single-track live-load is counted. The braking force amounts to 670 lb. per lin.ft. per truss. Wind was figured as 800 lb. per lin.ft. of bridge on the top chord, 700 lb. on the bottom chord, and 500 lb. on the train. An excess stress was taken into account in all members subjected to various kinds of stress, of an amount equal to the excess of wind and braking force over 20% of the sum of dead- and live-loads plus impact and lateral force. The impact allowance is as per Gustav Lindenthal's formula (Engineering News, Aug. 1, 1912). Secondary stresses and erection stresses received special investigation in relation to and because of the rigid gusset-plate connections.

Medium Steel

For this bridge a somewhat harder material is specified than ordinary commercial bridge steel. The specification calls for steel of ultimate strength 62,000 to 70,000 lb. per sq.in., which exceeds by 5000 or 6000 lb. the standard bridge steel of today. At the same time a bend test fully as severe as that required of bridge steel is demanded—180° cold bend flat.

Alloy steels at present prices showed no economy over carbon structural steel.

Erection

The bridge will be erected partly on falsework, partly by cantilevering out from the middle pier over the channel. An opening of 350 ft. has to be maintained in the channel for river traffic.

The bridge is being built by the C. & O. N. Ry. (G. W. Stevens, President; M. J. Caples, Vice-President). The design and detail plans of the bridge and approaches were worked out by Gustav Lindenthal, Consulting Engineer, New York, who is also in charge of the execution of the work. The foundations and masonry are being built by the Dravo Contracting Co., Pittsburgh. The contract for fabrication and erection of the steel superstructure has been let to the McClintic-Marshall Co., Pittsburgh, Penn.

Construction work on the foundations was started in November, 1914. It is expected that the bridge will be completed and ready for traffic by November, 1916.

Increasing Ontario Hydro-Electric Power—The total revenue from the Niagara hydro-electric system for the six months ending Apr. 30 was $710,324, according to returns just received by the Ontario Hydro-electric Commission from 84 municipalities, railways and large users of power in the Niagara district. Of this sum $353,508 was expended for power, $155,120 for interest charges, $49,696 for maintenance, $30,815 for operating expense and $1156 for engineering and auditing. Of the balance, $69,808 was devoted to the sinking fund and $50,221 to the depreciation reserve. At the end of April the total consumption of power under commission control was approximately 88,000 hp., of which Niagara contributed 75,281 hp., the Ottawa, Port Arthur, Big Chute and Wasdels Falls developing the rest. At the present rate of increase the remainder of the 100,000 hp. contracted for with the Ontario Power Co. will be required before the end of the year. Having in view the early exhaustion of the supply now under contract, the commission is preparing plans for submission to the Ontario government for the development of new power. It is understood that these plans will provide for an ultimate total development of 250,000 hp. and that the bulk of this will probably be derived from developments upon the Welland Canal and a proposed public development on the Niagara River. New schemes, however, are under consideration. Contracts for the construction of the distributing stations and connecting line of the Eugenia Falls development were recently let. Adam Beck, chairman of the Ontario Hydro-electric Commission, states that the territory covered by the operation of the system is being rapidly extended. During the last two months the commission has concluded contracts with 17 additional municipalities for supplying electricity, and 14 others have made application for it and will submit by-laws to their citizens for authorization to make contracts. The system is now operating 2200 mi. of transmission lines, outside of the local distribution systems of the municipalities, and an additional 200 mi. is under construction.

Useful New Facts About Materials

The following short articles have all been abstracted from papers, with discussions, presented to the annual meeting of the American Society for Testing Materials, held June 22 to 26, 1915.

New Testing Devices

Every year some new testing device or machine is brought forward. Three of the more important described at the meeting are a bin-pressure scale, a peculiar arrangement to study cylinder lubrication, and a Charpy impact machine. On p. 66 a well-designed wire extensometer is described.

Apparatus for Determining Pressures on Walls

A comparatively simple testing apparatus has been devised by J. Hammond Smith, of the University of Pittsburgh, by the use of which laboratory determinations may be made of the pressures on walls or in bins of any kind of filling. The device consists essentially of a vertical bin with one face hinged around its lower end and so connected to a series of multiplying levers and scales that the side pressure of the fill may be easily calculated in direction, location and magnitude. The arrangement is quite ingenious, but the limitations of size and conditions of fill would seem to detract from its practical value.

Lubrication-Testing Machine

A machine was described by A. E. Flowers, of Ohio State University, for testing engine-cylinder friction and the effect of different lubrication under varying conditions of piston-ring pressure, steam-pressure and temperature. In a long steam cylinder there were two pistons connected by a rod and having steam admitted to the space between them. A long driving rod, with ball-bearing end connections, led to a special electric motor with which close speed adjustment and accurate load determination could be made. The outfit included also a high-pressure superheating flash boiler.

It was found that to maintain constant friction required a liberal supply of lubricant, most of which was carried out directly with the exhaust. Reducing the supply of lubricant brought about an increase of friction within 5 min. Increasing the supply brought a reduction within 15 min.; but with low steam pressures or when starting with dry piston surfaces, it often took 2 hr. to reach a consistent low value. Continued increase of lubricant did not reduce friction below a certain minimum. The coefficients obtained were all large, 3 to 4 per cent. or more, and very much larger than the values found in well-lubricated bearings. The coefficients obtained and the amount of lubricant necessary were greatly modified by the presence of steam and by the difference of steam pressure on the two sides of the piston.

Impact Steel-Testing Machine

The Charpy machine, for chopping through small specimens of steel and measuring the energy absorbed at the break, was described by J. J. Thomas in the paper entitled "Charpy Impact Test on Heat-Treated Steels." A pendulum hammer with knife, capable of delivering a blow of 217 ft.-lb. energy, showed the work left in it after striking by recording the angle of upswing past the broken specimen. Experiments showed no effect of flaring the test-bar notch out to an angle of 30 deg. The tests cited appeared to distinguish degrees of brittleness, but four or more tests were advised to avoid discrepancies of single specimens.

Fusibility of Coal Ash

Excessive clinker troubles with some coals are bringing a growing demand for including in specifications for the purchase of coal a limit to the fusing or softening temperature of the ash. Tests to determine softening temperatures were reported by A. C. Fieldner, A. E. Hall, and A. L. Feild, of the U. S. Bureau of Mines, in the paper entitled "Fusibility of Coal Ash." Comparative studies were made of 18 types of coal ash in six different furnaces under various oxidizing and reducing atmospheres.

The ash was made up into three different sizes of pyramids, similar to Seger test cones. A slender type, $\frac{3}{16}$-in. base by 1 in. height, was the most satisfactory in giving shorter and more definite softening intervals and less trouble from evolution of gases. The most satisfactory indication was obtained by mounting the cone with one side vertical. Further inclination was undesirable and led to premature falling. When the ash had been ground to an impalpable powder the cones tended to soften at slightly lower temperature than when made with ash merely passing a 100-mesh screen. The slower rates of heating gave lower softening points; a rate of above 10 deg. C. per minute led to inaccuracy. Below 2 deg. C. per minute made the runs too tedious. It was recommended that a rate of 5 deg. C. per minute be maintained up to initial deformation and that then this be reduced to 2 deg. C. per minute.

The atmosphere in which the ash was heated proved to be the most important factor in varying the softening temperature. The highest softening points were obtained either in air or carbon monoxide; the lowest temperatures were obtained in a mixture of hydrogen and water vapor, ranging from 30 to 70 per cent. water vapor. Analysis of the cones which had been melted in an atmosphere of 50:50 per cent. hydrogen and water vapor showed that the iron oxide in the ash was combined principally in the ferrous state similar to the condition actually found in fuel-bed clinkers.

It was recommended that such tests should be carried out in an atmosphere of 30 to 70 per cent. water vapor in hydrogen, under conditions easily reproducible in different laboratories.

Deterioration of Bitumens

Samples of three gas-house tars, two coke-oven tars, two water-gas tars, one blast-furnace tar, one pine tar, one stearine pitch, two refined asphalts, one fluxed grahamite, one fluxed gilsonite, one straight-run California asphalt, one straight-run Mexican asphalt, one blown fluxed Texas asphalt and one straight-run fluxed Texas asphalt were exposed to the effects of overheating, "gas drips," illuminating gas and sunlight and air to study their deterioration. The results were presented by S. R. Church and J. M. Weiss in the paper entitled "Some Experiments on Technical Bitumens."

In studying the effect of overheating, 50 grams of pitch, or 45 grams of asphalt, were heated for 2 hr. in an open iron crucible, at 400 deg. F. for the pitches and 450 deg. F. for the asphalts. The pitches, excluding the blast-furnace tar and pine-tar pitch, showed an average loss of 6.2 per cent., compared with 3.8 per cent. for the asphalt. The increase in melting point of the pitches averaged 24 deg. The increase for the asphalt was 20 deg. The results indicated that moderate overheating had no serious effect.

"Gas drips" is a condensate from illuminating gas found more or less in all gas mains. It often leaks into the ground and destroys bituminous materials used in waterproofing foundations, etc. In studying its action on these pitches thin films (0.012-0.019 in. thick) of the bitumens were placed over perforated papers on special glass tubes. The tubes were immersed in a beaker of the drips so as to give a pressure head of 1 in. of liquid on the bitumen film. The time in minutes required for penetration was noted. The asphalts required from 1 to 4 min.; water-gas tar, 3 to 5 min.; the coke-oven and coal-gas tars, from 99 to 135 min.

In studying the action of illuminating gas, bitumen samples were maintained at 60 deg. F. in an atmosphere of gas for six days. There was an almost inappreciable gain in weight of the tar, while the asphalts took on from 2 to 4 per cent. There was an appreciable softening of some asphalts and of the water-gas tar pitches.

In making the air and sunlight exposures, the samples were exposed on petri dishes in a dustproof box with a glass top. Air circulation was provided by side vents filled with cotton and protected by a dashboard. They remained in this box on a roof for one year. The changes in the bitumens were not very marked, either in chemical or physical properties. The tar pitches showed an average increase in melting point of 3 deg. F. and the asphalts an increase of 16 deg. F. The pitches showed a decrease in penetration at normal temperature of 21 deg. and the asphalts a decrease of 23 deg. None of the bitumens showed any measurable increase in matter insoluble in benzol, but two gas-house tar pitches and one asphalt increased from 2 to 6 per cent. in matter insoluble in carbon tetrachloride.

New Test for Galvanizing

The paper entitled "Determination of Spelter Coating on Sheets and Wire," by J. A. Aupperle, proposed a new test for quantity of galvanizing to replace the Preece copper-sulphate and the Walker lead-acetate tests, the former being inaccurate and the latter requiring too much time.

The method hinges on using a small amount of antimony chloride in a hydrochloric-acid dip. The chloride appears to hasten the solution of the coating and, after the iron is exposed, plates a thin film on the base metal to retard attack. The small amount of antimony that plates out on the surface of the iron can easily be removed by scrubbing under running water.

For determining the weight of sheets several samples 2¼ in. square are cut from various parts. About five of these are weighed together and immersed singly for one minute in 100 c.c. of hydrochloric acid (sp.gr. 1.2) to which has been added 5 c.c. of an antimony chloride solution (20 grams in 1000 c.c. of hydrochloric acid). The dip can be used for the five samples, but 5 c.c. of the antimony liquid should be added for each sample on account of the metal deposited. Samples are washed and scrubbed under running water, dried with a towel and laid in a warm place for a few seconds. The samples are weighed together and the number of grams lost is divided by the number of pieces used. Each gram corresponds to 1 oz. of coating per sq.ft. of sheet (1 oz. per 2 sq.ft. of galvanizing).

In testing wire the length of sample should be chosen so that the number of grams of coating represent the number of ounces per square foot. (This varies from 12.1 cm. length for No. 0 gage to 49.5 cm. for No. 14 gage.) Only one-half the amount of antimony chloride is added to the acid on account of wire exposing one-half the area of plate. The coating on galvanized sheets, while expressed in ounces per square foot, considers metal on both sides of each square foot, in reality representing 2 sq.ft.

Surface Decarburization of Heat-Treated Steel

It is known that loss of carbon at the surface of heat-treated steel frequently exists and defeats the object of the treatment. J. G. Ayers, Jr., in the paper "Micrographic Determination of Surface Decarburization in Heat-Treated Steels," described a physical method of quantitatively studying decarburization. The steel under study was first heated to 1000 deg. in an atmosphere of nitrogen, so as to make the carbon-bearing portions visible without reduction of surface carbon content. The specimens were polished, etched and photomicrographed. From the prints the proportion of sorbitic pearlite present was estimated and the per cent. carbon deduced from that. The method was illustrated by study of one case where the mill process was a greater factor in surface decarburization than the heat treatment.

Molten Zinc for Etching Iron and Steel

A paper entitled "Molten Zinc as a Reagent for the Macroscopic Etching of Iron and Steel," by J. L. Jones, described a method for making visible, on a large scale, porosity, lack of homogeneity, cracks, blisters, seams, overlaps and other defects usually covered in rolling and forging. The scheme employed the action of molten zinc in dissolving highly carbonized steel.

Some typical tests were described. Bars of various materials were suspended in a galvanizing pot; on taking them out the dross was broken off with a hammer while hot and the coating of metallic zinc was removed by immersion in dilute sulphuric acid. Some specimens of high-speed tool steel had their surface deeply pitted and grooved, due to imperfect alloying of ferro-tungsten, etc. Several carbon-steel samples were eaten away remarkably; they showed also a curious shelly structure and apparently concentric layers of steel, as though the metal had not been sufficiently plastic to fill out the grooves in the mill rolls. A few sections of bessemer-steel rails which seemed perfectly solid before etching showed the web part of each sample to be spongy. A defective axle which failed in service showed a pipe. A rejected piece of die

steel showed a blistered portion deeply etched. A broken high-speed tool was not attacked on the hardened end, but the other was eroded.

Defective Battery Zincs

In the paper entitled "Battery Zincs; Some Causes of Defective Service," Robert Job and F. S. White described the physical examination of some poor battery zincs bought for the signal system of the Lehigh Valley R.R. Comparative chemical analyses of satisfactory and unsatisfactory shipments did not explain the trouble. Transverse sections were taken from several samples, polished and etched lightly with ammonium sulphide acidified with hydrochloric acid. The satisfactory zincs were fine-grained and of uniform structure; the unsatisfactory ones were coarse-grained and had a band of white metal around the outside. The inference was that they had been poured at too high temperature and that this had volatilized the mercury usually added to the metal.

Cracking of Copper Alloys

Further investigations of the service behavior of manganese bronze, naval brass, muntz metal and gunmetal was reported in the paper "Fatigue of Copper Alloys," by Ernst Jonson, New York Board of Water Supply. Tension-test specimens were stressed before and during corrosion.

The results of the investigations were generalized as follows: (1) Prolonged excessive stress does not injure the metal when it is protected from corrosion while under stress; (2) corrosion does not generally injure the metal when not accompanied by excessive stress; (3) corrosion accompanied by prolonged stress of less than 20,000 lb. per sq.in. does not generally injure the metal; (4) corrosion with ammonia accompanied by a prolonged stress of 20,000 lb. per sq.in. or more causes cracking similar to season cracking.

Ammonia appeared to be a representative corrosive agent, although it attacked these alloys more quickly than the average. Therefore, the ammonia reaction was regarded as an accelerated test, which in a few hours produces results that would ordinarily require days or months to develop. It was recommended that copper alloys in general should never be stressed beyond 20,000 lb. per sq.in. and that the process of manufacturing copper-alloy rods should be planned so that no appreciable initial tensile stress is left in the metal.

The Microstructure of Concrete

From a study through the microscope of the polished surfaces of opaque specimens of concrete Nathan C. Johnson has developed certain facts about the structure of concrete from which deductions as to the deterioration may be made. The facts and the deductions taken serially are: (1) The individual pieces of aggregates, both fine and coarse, in concrete are not in that close proximity one to the other that the void-filling theory (that is, that the aggregates are so proportioned in regard to voids that they all join together in an approximately complete solid) would predict. In consequence, too great a part of concrete is made up of the cement matrix, the weakest element in the mixture. (2) Large quantities of occluded air, in relatively small bubbles widely dispersed, reduce the actual concrete volume in a mass of concrete, with consequent reduction in possible strength. (3) Aggregations of solid particles commonly observed under the microscope and obviously not sand nor individual particles of cement appear to be unhydrated cement, thrown into groups because the surface tension in the water at the time of mixing prevents the dispersion of water to all the cement particles; in consequence the little groups of unhydrated cement are surrounded by shells of hydrated cement which the water did reach. These unhydrated portions are zones of danger because of possible future crystallization and expansion if water ever does reach them. Such crystallized zones are observable under the microscope. Experiments to discover some means of reducing surface tension as a remedy are incomplete and unsatisfactory, though Mr. Johnson considers such a remedy within the range of possibility.

In the discussion H. L. Spackman disagreed with Mr. Johnson in regard to the last-mentioned observation. He does not consider that the groups of unhydrated cement were segregated by the behavior of the water under surface tension but are merely the larger particles of cement. Groups of the same appearance can be observed in any normal clinker. This, if true, would destroy the value of any method of reducing surface tension. Mr. Spackman contended that the void tests for aggregate should be made on a wetted material from which water had been drawn by a vacuum pump, as an approximation of the conditions in concrete-making. He also doubted Mr. Johnson's zones of crystallization, saying that in all probability they were mere shrinkage cracks. P. H. Bates agreed with Mr. Spackman, and further pointed out that an extremely wet concrete, mixed much longer than is common, might avoid some of the troubles discovered by Mr. Johnson. Cement in a concrete mixture will continue to take up water of hydration for a very long time.

An Extensometer Rig for Wire Tests

In the laboratory of the Westinghouse Electric & Manufacturing Co., East Pittsburgh, Penn., tension tests of 240-in. lengths of wire are made with extensometer readings to 0.0001 in. T. D. Lynch described the extensometer arrangement in a paper on "Elastic Limit." The wire is suspended vertically in a 2-in. iron pipe, the lower end of which branches into two parallel pieces of pipe to which a cross-bar is clamped. An insulated micrometer screw mounted in this cross-bar can make electrical contact with a steel ball fixed in the bottom of a scale-pan hung to the wire and serving to apply load to the wire by laying weights in the pan. The micrometer screw has a large dial which reads direct to 0.0001 in. Before test, the wire is annealed and straightened by hanging a weight to it and heating the wire electrically until permanently straight. During the test the temperature of pipe and wire is maintained approximately constant by passing a gentle current of air down through the pipe.

With this rig a tension test on copper wire can be made to give an excellent straight line up to the limit of proportionality.

[*Further abstracts of a similar nature will appear in an early issue.*]

Deep Bridge Piers Sunk Without Air

***SYNOPSIS**—Difficulties encountered in placing concrete foundations for a bascule bridge of D., L. & W. R.R. at Buffalo, N. Y. Six piers sunk 70 ft. through five varying soil strata; open-cylinder method attempted in all cases, but resort to air necessary in two.*

In connection with extensive river improvements which are being carried on by the city at Buffalo, N. Y., the Delaware, Lackawanna & Western R.R. is required to reconstruct its bridge over the Buffalo River and to provide a bascule lift span. At this point an extensive piece of track-elevation work is in progress whereby the grade crossing with the Buffalo Creek R.R., the Pennsylvania R.R. (Chautauqua Division), and the Nickel Plate R.R., is being eliminated. The tracks of the Lackawanna are raised 22 ft. at the river and 26 ft. at the crossing of the railroads named. The reconstruction of the bridge has nothing to do with the track elevation aside from the fact that it is located within the limits of the grade-crossing elimination work.

The City of Buffalo is deepening the Buffalo River and, at the site of the bridge, slightly altering its course. This improvement was started in 1911, and the new bridge had to be completed this summer so as to be ready when the river improvement reached it. The width of the improved channel will be 190 ft. at the bottom, with 256 ft. between dock lines.

The depth of channel below mean low water will be 23 ft. Mean low water is elevation —3 with respect to city datum, and the normal difference at present between mean low water and mean high water is about 9 ft. This difference will be less after the enlarged channel is completed.

The new bridge is located substantially on the site of the old one, the west abutment of the new structure will be located at the center pier of the old one, and the removal of this pier was included in the foundation contract. Originally, the track centers were 13 ft. In order to permit the construction work, both at the bridge and throughout the elevation zone, to be carried out without interruption and still maintain certain important track connections, it was found desirable to spread the main tracks to 83-ft. centers, each track being moved out the same distance from the center line. The old bridge, consisting of four single-track through-girder spans 116 ft. 6 in. c. to c. (two for each track), was shifted accordingly and carried on pile bridgeseats. These girders were originally designed and fabricated so that they can be used either as through or deck girders. They will be used eventually as deck spans in the completed structure. Track centers on the completed work will be 15 ft.

The New Bridge

The new double-track bridge will consist from east to west of a deck span 42 ft. long c. to c.; a tower span 36 ft. long; a Strauss trunnion bascule lift span 125 ft. c. to c., giving a clear channel of 110 ft.; one deck span 116 ft. 6 in. long, composed of the girders from the old bridge, as noted; a deck span 44 ft. long c. to c.; a deck span 116 ft. 6 in. long. The last-named span is temporary and is required in order to maintain the present channel until the river improvement is completed, after which it will be removed and the embankment extended across the old channel.

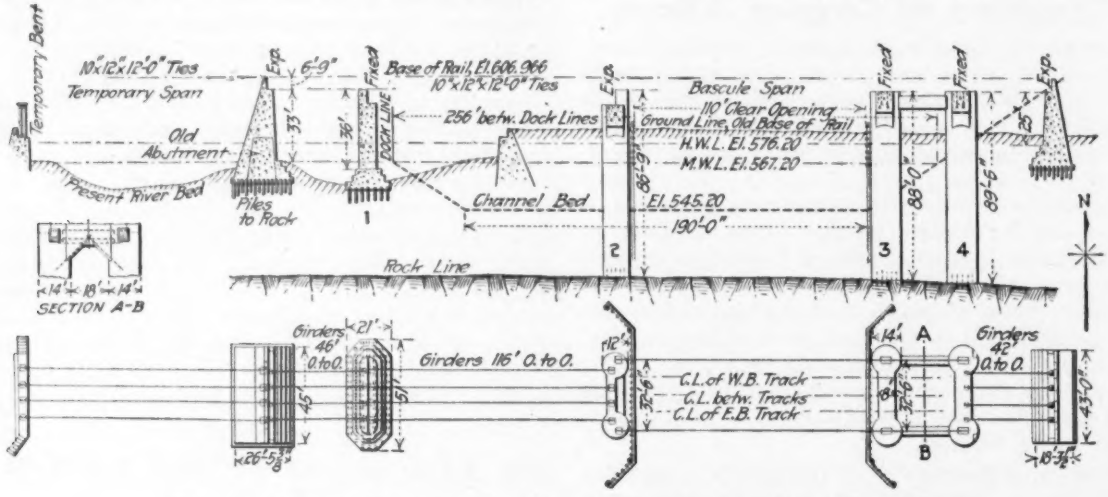


Fig. 1. Masonry Plan of New Bascule Bridge of D., L. & W. at Buffalo

Soil Conditions

Borings were made on both sides of the river as near as possible to location of piers and abutments. Rock was found to be about 70 ft. below the ground (track) level with five soil strata intervening (Fig. 2). First came 5 to 6 ft. of fill (railroad embankment); then about 13 ft. of alluvial material containing some butternut logs; third, a sand-and-gravel stratum of approximately the same depth. The lower part of this stratum was very compact. The fourth stratum was a reddish clay about 25 ft. deep completely impervious to water, but fully as plastic as soft putty. Last came a stratum of variable material: Very fine and tightly packed sand; sandy clay; hard red clay; very plastic red clay; and immediately overlying the rock from one to two feet was a hardpan of finely pul-

verized limestone which, under great pressure, had taken on the character of hard shale. In this last stratum were found boulders of sizes varying from one-man stone to 5 cu.yd. Some mention of these boulders will be made later.

Abutments and Pier 1

Because of the character of the material to be encountered, as determined by test borings and data secured from the piledriving for temporary structures, it was decided to carry all piers except Pier 1 to rock at a depth of 70 ft. The abutments, being of the buried type, are carried on piles driven to rock (53 ft. below cutoff at elevation —3) and loaded to a maximum of 18 tons each. The driving for the east abutment was done without incident. At the west abutment some difficulty was experienced because of boulders encountered, and eight or nine additional piles were driven on this account. When the work is completed both abutments will be buried as shown in Fig. 1.

Work first started on the east abutment and the sheetpiling for the coffer-dam was in place June 14, 1914; excavation was commenced six days later. This abutment was completed Aug. 23. An orange-peel bucket on a 20-ton locomotive crane was used to take out all the excavation except at Pier 1 and the west abutment, which are located in the present river channel. The spoil was loaded on 20-yd. side-dump cars running on a temporary track, and was hauled out and used in the fill for the track elevation a short distance from the bridge.

The coffer-dam and foundation piles for the west abutment and Pier 1 were put down by a floating driver. This portion of the work was commenced July 6, 1914. The scow supporting the driver was built on the job by company forces, of material from an old Howe truss. It was originally constructed and used to drive the piles supporting the old bridge in its temporary location. The foundation piles were driven, using a follower, before the coffer-dam was constructed. The coffer-dam of Wakefield sheeting was driven 2 ft. below the gravel stratum as shown by the test-borings. With tight sheeting, therefore, the foundation pit should have been free of water.

On account of the very compact character of the lower portion of this stratum some difficulty was experienced in putting down the sheeting, and considerable leakage occurred. All these leaks were closed with plank-and-canvas patches and by damming off with clay taken from the cylinder excavations. The most serious leak occurred on the north side of the coffer-dam for Pier 1, where a buried log was encountered. The sheeting was forced so far out of line that this leak could not be controlled, and a second line of sheeting (Lackawanna steel sheetpiling) was driven around it. After all the large leaks had been closed off, water still appeared in such quantities that a 6-in. centrifugal pump was required to keep each of these two coffer-dams dry. In placing concrete the water was prevented from washing over the concrete by means of sluiceways.

A 15-ton derrick equipped with a 60-ft. boom was set on piling between the west abutment and Pier 1, and was used to remove the old stone pier, handle the excavation, place concrete, and to pull the sheetpiling of the coffer-dams. This derrick was handled by means of a hoisting engine set up on the east bank of the river.

There are four piers and on the drawing they are numbered from west to east. Pier 1 is solid and rests on

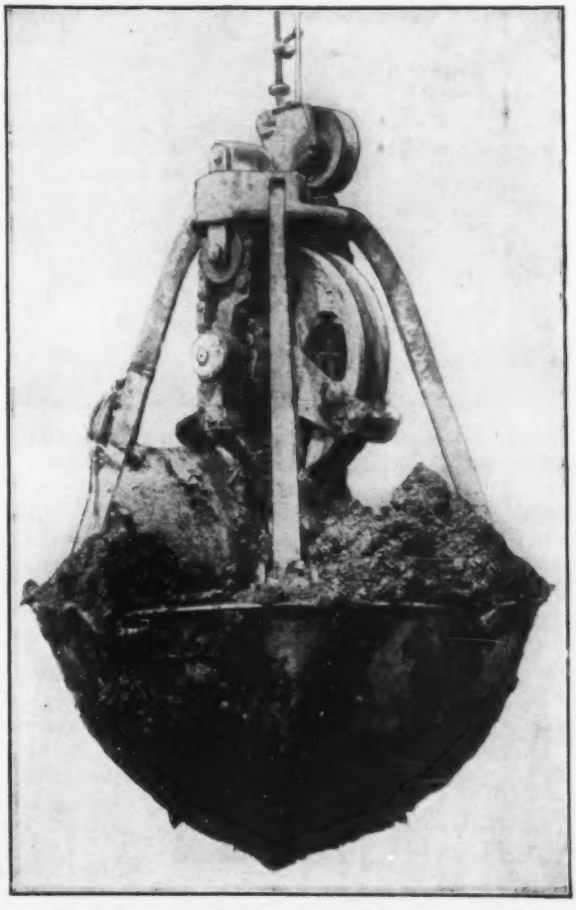


Fig. 3. Bucket Used for Dredging Cylinders
Especially heavy (about 5000 lb.) "Multi-Power" orange-peel bucket built by the Hayward Co., New York

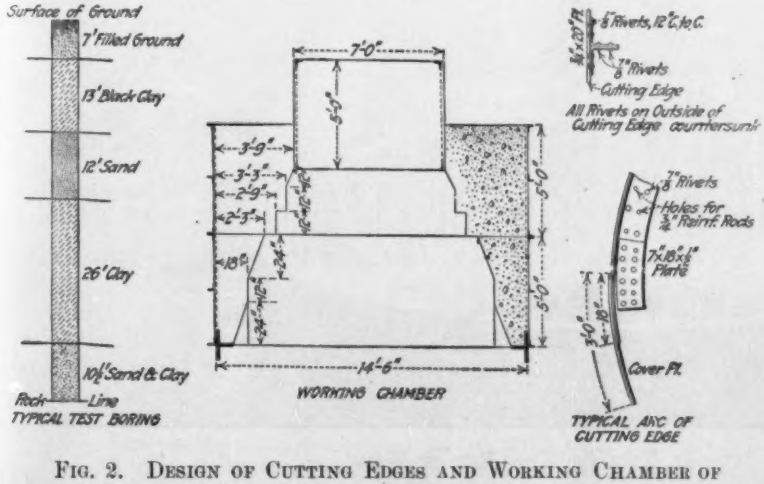


Fig. 2. Design of Cutting Edges and Working Chamber of Dredging Cylinders

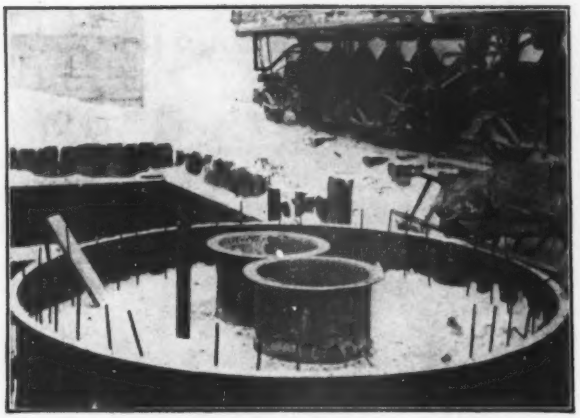


FIG. 4. A CYLINDER ON ROCK WITH AIR LOCK REMOVED
Compressed air was used on two of the piers

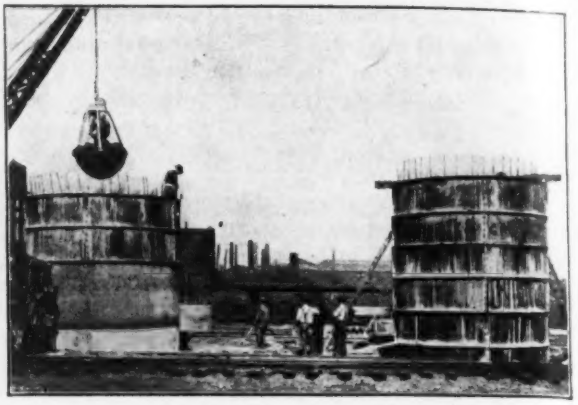


FIG. 5. REMOVING SPOIL FROM CYLINDER OF PIER 4 NORTH
North cylinder piers 3 and 4; method of excavating; 20-yd. dump car at extreme left

piles. Each of the others consists of two concrete cylinders (designated as north and south) tied together at the top with reinforced-concrete struts. The cylinders of Pier 2 are 12 ft. in diameter, and the cylinders of Piers 3 and 4 are each 14 ft. in diameter. They extend to rock 92 ft. below base of rail, 87 ft. below bridgeseat of the movable span, and 70 ft. below ground (present-track) level. Only the west abutment and Pier 1 stand in water; the other piers and the east abutment will not be wet until the channel is changed. Work on Piers 2 to 4 commenced in numerical order, Pier 2 being started shortly after the work was under way on the east abutment.

The open-cylinder method was used on all these piers. Cylinders 3 North and 4 South had to be finished by using compressed air as noted below. The shaft in all cylinders was 7 ft. in diameter; the arrangement of cutting edge and working chamber is shown in Fig. 2.

In starting to sink the piers the earth was excavated to a depth of about five feet, and the sides of this excavation were carefully cribbed with heavy timbers. The

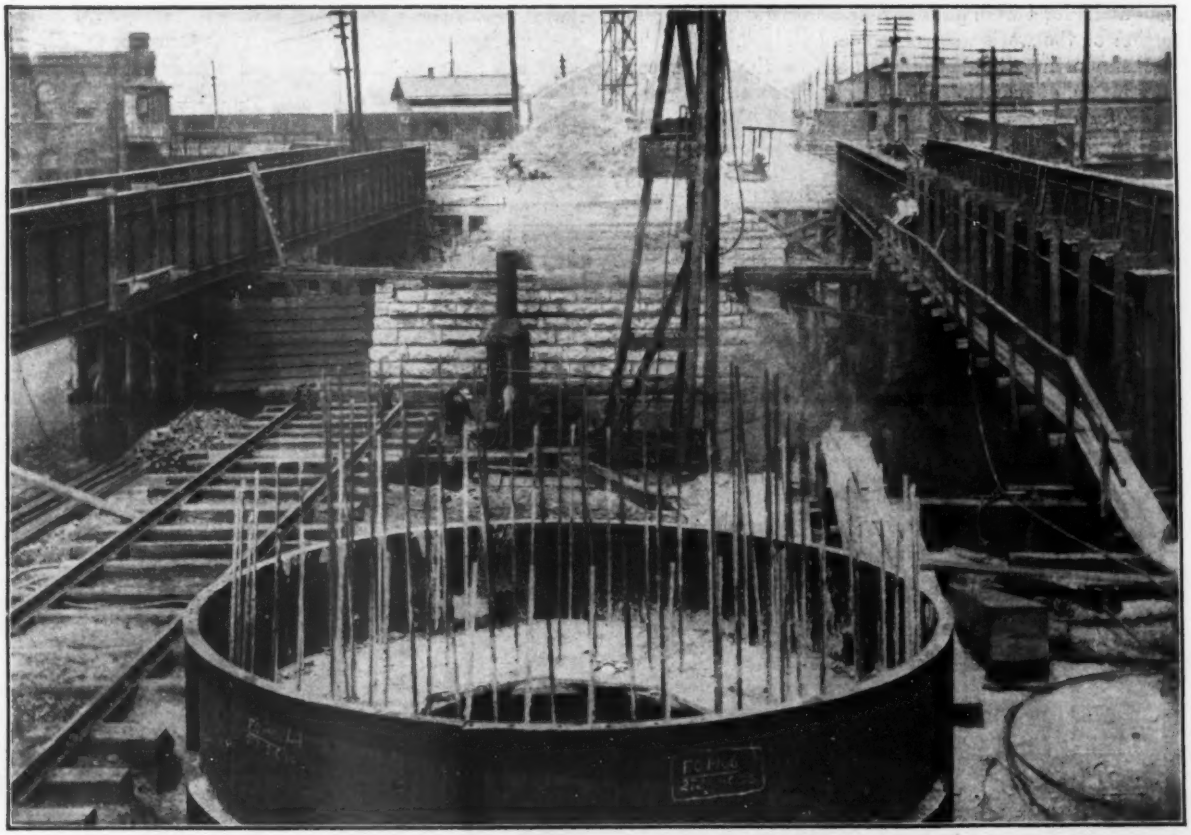


FIG. 6. GENERAL VIEW OF WEST END OF WORK
Cylinder of pier 2 north in foreground; center pier of old bridge not yet removed; manner in which tracks were spread and carried across Buffalo River

purpose of this cribbing was to brace the cylinders and assist in guiding them down in a vertical position, and without lateral shifting. On the bottom of this excavation the cutting edges were placed and carefully centered and blocked level. The interior forms (wood) of the working chamber were shaped as shown in Figs. 2 and 7 in order that the sinking of the cylinders could be kept under control. An outer row of ¾-in. round, smooth, vertical reinforcing rods were placed with the lower ends through the outer holes in the flange of the cutting edge (Fig. 2). This reinforcing was continued up to the bridge seat. Horizontal reinforcing rings of ¾-in. rods spaced about 12 in. apart were also provided. An inner row of vertical reinforcing rods bent over to conform to the slope of the inner face of the cutting chamber was also provided. These rods were fitted in the inner line of holes and were carried up only 16 ft.

After the cutting edges had been placed, the wood form for inner face of cutting chamber constructed and reinforcing placed, the exterior steel forms were set up and the whole again carefully leveled. A section of cylinder 10 ft. high was poured. As the cylinder was quite top-heavy at first the 10-ft. section was sunk by hand, better control being thus maintained. The earth was removed by the orange-peel bucket mentioned. After water was encountered all excavation was done by the locomotive crane. The cylinders were all constructed and sunk in 10-ft. sections.

Neither cylinder for Pier 2 encountered any obstructions and went down without difficulty until about two feet off rock. Sufficient material could not then be excavated to obtain settlement and this had to be secured by loading them. Their cutting edges were laid July 6, 1914. On Aug. 29 each was two feet off rock. They remained some days without appreciable settlement; then they were loaded with stone removed from the old center pier. The north cylinder remained loaded until Sept. 20, 1914, when it was one foot off rock. It was then carefully cleaned out and filled with concrete. The south cylinder settled the same amount by Oct. 18, when it was also cleaned out and filled.

The north cylinder of Pier 4 was started Aug. 1, and was filled Nov. 2. This cylinder was sunk without incident, save that at one dredging considerable difficulty was experienced in getting it to start. When it finally did start to settle, it sank until the top of the cylinder was below the bottom of the excavation.

The north cylinder of Pier 3 was started at the same time as the north cylinder of Pier 4. At a depth of about 20 ft. (about 5 ft. in the gravel stratum) a large butternut log was encountered, which interfered with the operation of the orange-peel; but some excavation was taken out and finally the cutting edge cut through the log, which was then taken out in pieces. About 8 ft. above rock a very large boulder was struck. The cylinder was pumped out, but because of the plastic character of the soil, the rock could not be uncovered sufficiently to do any effective job of breaking it up. Well drills were used outside the cylinder, and the boulder was broken up with a heavy charge of dynamite. After this was accomplished and the fragments removed, the soil rose about 25 ft. in the cylinder and could not be pushed back. Eventually, it was found necessary to apply air. Before the air lock was applied the cylinder was filled with water and the earth excavated to the level of the cutting edge. The air was then applied and the water bailed out. Dredging then proceeded successfully and the pier reached rock on Dec. 26.

The south cylinders of Piers 3 and 4 were started about the same time, Aug. 8, 1914, and went down to a point about 12 ft. above rock without any special incident. At this point Clyinder 4 South stopped and did not settle further. Cylinder 3 South eventually continued down to a point 5 ft. above rock and stopped. Continued excavation had no effect on it, and it eventually developed that a seam of exceedingly plastic material had been encountered. After about 150 cu.yd. had been removed excavation was discontinued. This was on Nov. 3, and about

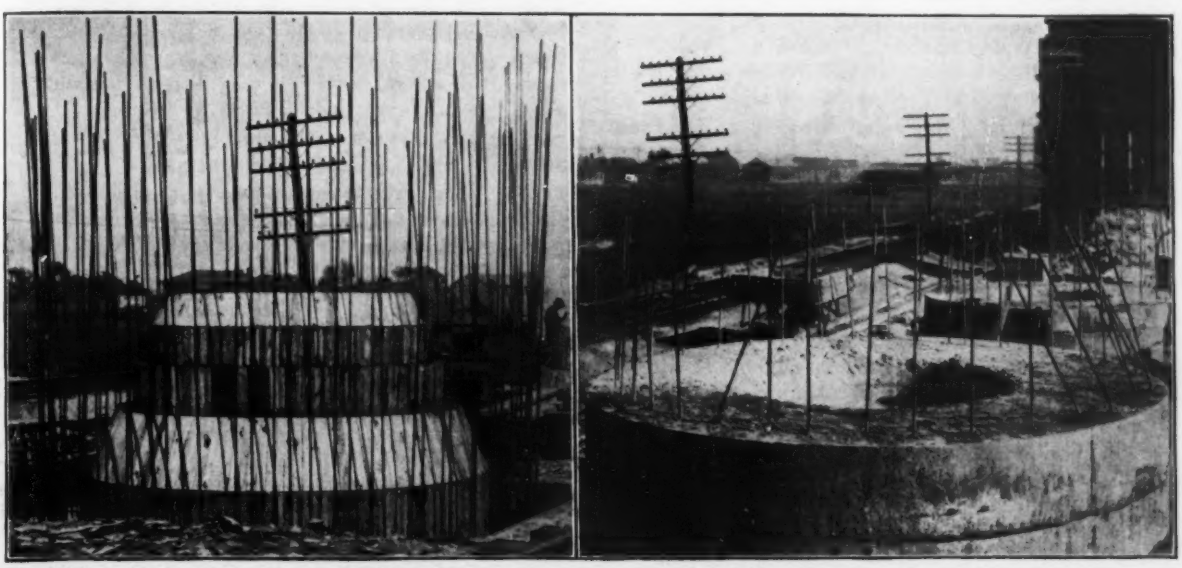


FIG. 7. TWO VIEWS OF PIER WORK

Left—Form for cutting chamber of a cylinder, showing vertical reinforcing rods for first section. Right—Piers 3 and 4 north; east abutment partially completed; westbound main track and construction tracks beyond piers

noon a slight subsidence of the ground in the vicinity of the cylinder was noticeable. By night this amounted to 5 in. No further subsidence occurred until Nov. 6, when a further settlement of 18 in. took place. After this the subsidence was gradual, reaching a maximum of 9 ft. by Nov. 8. The main tracks were affected and considerable work was required to keep them in serviceable condition. The most serious effect, however, was that the movement of the earth toward the point of greatest settlement caused the tipping of the south cylinder of Pier 4. The cylinder apparently pivoted on the very compact layer of the gravel stratum and the top moved to the northwest so it was out of position 135/8 in. to the west and 91/8 in. to the north. The bottom moved in the opposite direction an amount nearly double that of the top. This was the correct proportion if it pivoted on the gravel stratum.

While waiting for the air equipment, Cylinder 3 South settled to a point 18 in. from the rock. By careful handling, it was found possible to complete the excavation and dam off the flowing material and clean out and fill with concrete. This work was completed Nov. 28, 1914.

Immediately after this cylinder was filled heavy timber braces were placed between Cylinder 4 South and Cylinder 3 South and 4 North. In addition to the braces 35-ton jacks were also used, and the whole system of bracing continued until the cylinder straightened up. When ready to apply the air the cylinder was filled with water and the soil removed to the bottom of the cutting edge as in Pier 3 North. As the cylinder showed some disposition not to settle, it was loaded with 90 tons of pig iron. The loading was eccentric in order to assist the process of straightening. After the air was applied the cylinder sank without difficulty and as it settled it straightened up, and is now plumb. The cylinder was filled Jan. 25, 1915.

Twenty pounds' air pressure was used. The air lock used was furnished by the Union Iron Works, Hoboken, N. J., in 14 days, from a special design furnished by the contractor. The cutting edges and steel forms were furnished by the Lackawanna Bridge Co. The work was done under the supervision of G. J. Ray, Chief Engineer of the Delaware, Lackawanna & Western R.R., A. E. Deal, Bridge Engineer, and George E. Boyd, Division Engineer, by Walter H. Gahagan, contracting engineer, Brooklyn, N. Y. (Boyd C. Myers, Superintendent on the job.)

Superelevation of Curves on Highways, Illinois Practice

By H. E. Bilger*

The advent of the motor vehicle upon our rural public highways, with the consequent stimulus that has been given to the adoption of types of highway improvement best suited to withstand this new character of traffic, presents to the engineer many details of design and construction that heretofore have not required much consideration.

One of the details that are demanding more and more attention each year is the matter of the proper crown, cross-slope or superelevation of the pavement on curves. As in most other matters of engineering design or construction, there can be no one best solution of the problem, inasmuch as the governing conditions on different pieces of work may bear relatively different weights.

Generally speaking, I would say that the treatment to be administered should vary with the type of the pavement and with the relative amounts of motor and horse-drawn traffic, and should depend upon such minor considerations as the radius of the curve and the climatic conditions.

This matter is receiving especially careful consideration in Illinois, inasmuch as for practically all state-aid road construction vitrified brick and portland-cement concrete alone have been adopted up to this time. The employment of these costly types of pavement requires for the full development of their merits that all matters relating to the horizontal and the vertical alignment of the pavement be cared for just as accurately as though the work was on a city street.

In Illinois the popular widths of brick and concrete pavements are 10 and 18 ft., though the 15-ft. width is destined soon to surpass these in point of mileage. For other types of construction the prevailing widths are 10, 12 and 15 ft.; but the 12-ft. width is losing in popularity. On tangents the brick and the concrete pavements are designed with a 1-in. crown for the 10-ft. widths and with a 2-in. crown for the 18-ft. widths.

From past experience it would seem that, all matters considered, the most satisfactory treatment of these types on curves is to carry the profile of the center line and of the inner edge of the pavement around the curve without a break, but to elevate the outer longitudinal half of the slab 2 in. for the 10-ft. pavements and 4 in. for the 18-ft. ones. On the inner as well as the outer longitudinal half of the slab the convexity of the surface should be avoided, to the end that the entire portion of the slab that comes upon the curve may be a surface having a straight-line top on any cross-section.

Gravel and macadam road surface should be similarly treated on curves—that is, by elevating the outer longitudinal half of the metaled way by an amount equal to twice the crown on tangents for the particular type in question.

Where the central angle of the curve is more than about 30 deg., the width of the pavement around the curve should be increased by some 30 or 40 per cent., and the pavement's position should be such as to bring its inner edge to within some 4 ft. of the fence corner, necessitating the inner ditch being converted into an underground drain.

When such pressure is brought to bear by the tax-paying public as to require the engineer to construct a brick or concrete pavement to one side of the center line of the traveled way, the upper surface of the pavement should be flat and, of course, should drain to the nearest side ditch.

Occasionally theoretical arguments are advanced for crowning, or rather convexing, the upper surface of the pavement that drains to one side only. The argument usually put forth is that if the completed surface is not convex it will present the unsightly appearance of being concave. This is merely a matter of construction. If the design of the template is such as to assure against deflection, I have not yet heard advanced a worthy argument for a brick or concrete "off-center" pavement with a convex surface.

*Road Engineer, Illinois State Highway Department, Springfield, Ill.

In anticipation of a deflection of the template it would, of course, be only the part of wisdom to use a template curved convexly by at least the amount of the expected deflection; but the use of a template of questionable rigidity is clearly not in accord with the other construction requirements of good brick and concrete pavements.

San Francisco Convention of the National Electric Light Association*

San Francisco has been the natural rendezvous this year of many and varied organizations, so that it is a significant fact when a particular society can make itself conspicuous in the busy life of the Exposition City. Thus far no other visitors probably have been so singularly honored as were the members of the National Electric Light Association, which held its thirty-eighth annual convention here June 7 to 11. An imposing and beautifully illuminated colonnade was specially built for them in the downtown section; at the Exposition grounds a night was appointed in their honor for exceptionally spectacular fireworks and illumination effects.

The night wonders of the Exposition are essentially electrical, and every deference was shown to the men of the industry which made them possible; on the other hand, the Exposition was particularly pleasing to the convention because of the acknowledged triumph of its illumination as a beauty feature, and on every hand warm praise was given to the designer, W. D'A. Ryan, of Schenectady, N. Y.

Association Work

The Committee on Organization (George Williams, of New York City, chairman) reported a total of 13,448 members—an increase of 431. The treasurer (W. F. Wells, of Brooklyn, N. Y.) reported an annual income of $136,327, expenses of $122,811, assets of $43,667, and liabilities of $9420. Expenses and income of the convention were $29,000 and $27,000 respectively.

The chairman of the Committee on Education, John F. Gilchrist, of Chicago, gave a brief outline of the committee's activities. It was intended to have leading men give lectures before student bodies, and likewise to have some of the prominent university men do the same before the association. It was purposed also to establish courses for ambitious college graduates desiring to enter the electrical industry, and especially to provide vacation work for students. There were reported 262 students spoken for, most of the work being in the commercial departments, where the students become directly profitable to the companies employing them. Complete records of the fitness and capacity of each student will be filed with the committee.

"Electrical Prosperity Week," to be held by the Society for Electrical Development Nov. 29 to Dec. 4, was explained to the association. This will be the closing week of the Exposition, and every effort will be made to make the week a memorable one. It is aimed to illuminate and decorate every city and town in the country.

The annual report of the Committee on Progress (T. C. Martin, of New York City, sole member) stated that within the last 30 years the net earnings of public utilities had been 8.5 per cent., compared with 4.3 per cent. for railroads and 7.8 per cent. for industrial concerns. Risk of receivership had been 0.37 per cent. for public utilities, compared with 1.84 per cent. for railroads and 2.1 per cent. for industrial concerns. The increase of net earnings of gas and electric companies between 1907 and 1911 was 60 per cent., compared with 5 per cent. for steam railways.

The report discussed the status of municipal ownership, stating that while the total number of electric municipal stations in this country were 29.9 per cent. of the total, they earned only 7.7 per cent. of the gross income of the industry, which was $302,116,000. The municipal plants had 18.2 per cent. of the estimated total number of arc lamps.

Public Policy

The report of the Public Policy Committee (W. W. Freeman, of Cincinnati, chairman) commented in some detail on certain commission decisions, which seemed to show greater liberality toward utility corporations in the rate of return, protection from competition, etc. It was stated that there had been no final determination of law, by any high court in any state, fixing rates for electric service—all the decisions being in cases of gas, water and transportation concerns. Protection from competition was discussed and arguments were presented for regulated monopolies.

In a paper entitled "Municipal Regulation of Public Utilities" J. H. Roemer, chief counsel of H. M. Byllesby & Co. and former chairman of the Wisconsin Railroad Commission, gave a lengthy justification for state control of utilities being preferred to local regulation. He charged municipal electric plants with poor service in general, with being slow to respond to new discoveries and holding to crude methods, with discrimination in rates as flagrant as in private corporations, and with poor bookkeeping and office systems.

"Public-Utility Regulation in California" was the title of a forceful address by Max Thelen, president of the California Railroad Commission. One of the difficulties he described was in regulating rates where one company supplied as many as 175 cities and towns with various classes of service. He also reviewed the policy of the state in the matter of extensions which would have to be made in all fields which the companies claimed, before any protection against competion could be given. The commission aims to safeguard utility securities so that an investor anywhere in the country may be reasonably safe. This has helped to promote the sale of local securities within the state. Several millions of dollars in stock of the Pacific Gas & Electric Co. have been so disposed of at a cost of 65c. per share, compared with the 50 per cent. discount which at times had been demanded by brokers for selling securities.

New Officers

The following officers were elected for the ensuing year: President, E. W. Lloyd, Chicago; vice-presidents, H. A. Wagner, Baltimore; W. F. Wells, Brooklyn; R. H. Ballard, Los Angeles, and R. S. Orr, Pittsburgh; treasurer, W. H. Atkins, Boston; secretary, T. Commerford Martin, New York.

*Reported by E. Owen McCann, Humboldt Bank Building, San Francisco.

Field and Office

Loading Test of Lagged Piles in Soft Silt

By C. W. Staniford*

In Engineering News, July 23, 1914, in a view similar to Fig. 1, there were shown two sets of foundation piles loaded with concrete blocks as a test for an actual loading in the soft silt of the North River at New York City. The pier being built at this site (West 46th St.) presents two opposite conditions of foundation, in that

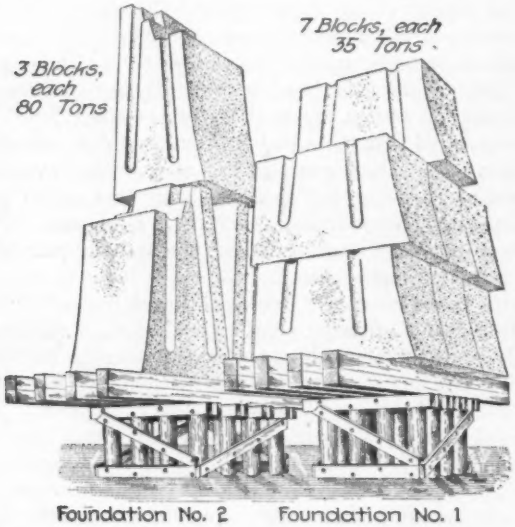


Fig. 1. 230-Ton Pile Tests in North River

the inner end is being closed by a sheetpile coffer-dam inside of which the solid rock is to be blasted to a depth of 44 ft. below mean low water; while extending from the coffer-dam outshore the solid rock gradually dips so that at the pierhead line, 1000 ft. from the bulkhead line, the rock is 150 ft. below mean low water. The overlying material is a soft silt furnishing no bearing capacity for an ordinary pile of the longest length.

Experience in founding the columns for double-deck steel sheds with their heavy loads indicated that the standard lagged piles, arranged in groups and capped with the form of grillage generally used, would perform the required service with safety. However, in order to produce these foundations at a minimum cost and to reduce the expense of lagging, it was the desire for this pier to create a foundation with as few lagged piles as possible and to arrange them in the most economical manner to produce the best carrying result. Therefore the first foundation, called "Foundation No. 1," was designed and placed so that every other pile was provided with four pieces of lagging, consisting of 5x6-in. sticks, 30 ft. long, bolted to the pile as shown in Fig. 2.

On account of the large settlement in this foundation, another group of piles, each pile of the group being provided with lagging and the spacing being increased so as to spread the load over a larger area, was designed and subjected to test. This is labeled on the drawing, "Foundation No. 2."

Each foundation was subjected as soon as built to the maximum theoretical load of about 230 tons, the loading being made with concrete blocks to be used elsewhere in the harbor. This second foundation also showed large settlement in the first 60 days, but thereafter the settlement became less than that shown for Foundation No. 1, until at the completion of the test, when both appeared to have come to rest, there was a difference in settlement of about 3 in. The total settlement for Foundation No. 1 was about 11¼ in. and for Foundation No. 2 about 8¼ in. The log of the settlement is given in Fig. 3.

Inasmuch as the load which these foundations have to carry will be gradually applied during the construction of the pier and the erection of the steelwork for the shed, and also as the maximum theoretical load may never be realized except possibly for short intervals of time, it was expected that these fully-lagged piles would become so thoroughly set and incorporated in the supporting bottom that little, if any, settlement would occur if the

*Chief Engineer, Department of Docks and Ferries, New York City.

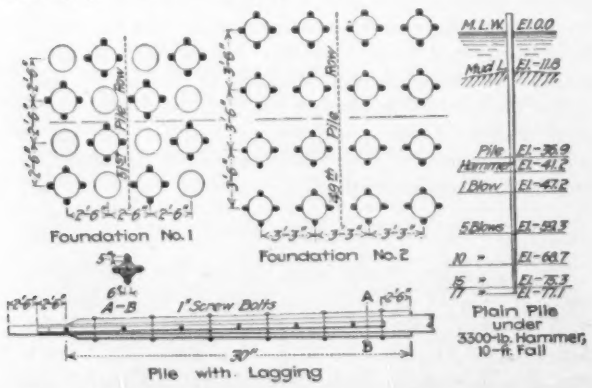


Fig. 2. Layouts of Pile Tests and Details of Piles

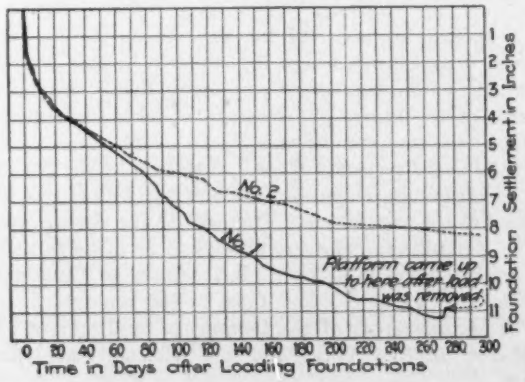

No. 2
No. 1
Platform came up to here after load was removed.
Foundation Settlement in Inches
Time in Days after Loading Foundations

Fig. 3. Log of Pile Tests

foundations were designed as shown on the diagram for Foundation No. 2. Accordingly, all column foundations for this pier requiring long piles in soft bottom were thus designed; that is, using fully-lagged piles with the larger spacing between piles.

Before having the benefit of these actual long-time tests to a quiescent state of settlement, the department constructed the nine Chelsea Piers in similar material, and at the present time, after about seven years of use with the heaviest loadings, the settlement is not discernible with the eye; there has been, however, an actual settlement of about 3 in. detected by instrumental observation. As the result of this recent test and the use of grillages as outlined in Foundation No. 2, it is expected that the settlement in the 46th St. Pier will be less, and in any event that it will be symmetrical.

The 46th St. Pier is being constructed by Holbrook, Cabot & Rollins, Inc., for the Department of Docks and Ferries of New York City, under the direction of R. A. C. Smith, Commissioner, R. C. Harrison, First Deputy Commissioner, and C. W. Staniford, Chief Engineer. A. J. Bryson is in charge for the contractors.

Three Points in Chimney Design

From a standard specification for perforated radial-brick chimneys prepared for its clients by the M. W. Kellogg Co., 140 Cedar St., New York City, the following three items of instruction and explanation are selected as being of especial interest:

Lining for Brick Chimneys.—The height of the lining for chimneys is found in the following manner: For ordinary conditions, where the temperature does not exceed 800 deg. F., the lining should be approximately one-fifth the height of the chimney; above 800 deg. and below 1200 deg., one-half the total height; above 1200 deg. and below 2000 deg., full height.

Protection against Lightning.—For lightning protection two points are the minimum for any chimney of diameter up to 5 ft. Above this, one point should be added for every 2 ft. in diameter or fraction thereof. The points of the conductor should be of copper ¾ in. in diameter by 8 ft. long, with a 1½-in. platinum-covered tip. They should be anchored at the top of the column and extend from the bottom of the corbeling upward. The lower ends of the points are connected by a copper cable which encircles the chimney. From this loop a 1½-in. 7-strand No. 10 Stubs' gage copper cable is carried down the side of the chimney and connected to a copper ground plate of the three-winged type. On the way up, the cable is anchored every 7 ft. with brass anchors, which support the weight of the cable. The ground plate is buried by the foundation contractor at the time the foundation is built.

Wall thicknesses of chimneys are obtained from the accompanying diagram. The table shown below is offered for use in quickly determining the maximum outside diameter of the chimney at the base, for various heights and inside diameters.

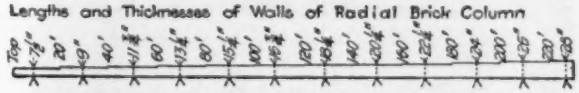


TABLE OF BOTTOM DIAMETERS OF RADIAL-BRICK CHIMNEYS

Height of Chimney in ft.	Internal Diameter at Top														
	3'	3'6"	4'	4'0"	5'	5'6"	6'	6'6"	7'	7'0"	8'	8'0"	9'	9'0"	10'
	Diameters in Feet at Bottom of Column														
75	7.42	7.69	7.96	8.46	8.96	9.46	9.96								
80	7.80	8.04	8.27	8.70	9.13	9.58	10.02								
85	8.18	8.38	8.58	8.95	9.31	9.70	10.08								
90	8.57	8.73	8.88	9.18	9.48	9.81	10.13								
95	8.95	9.07	9.19	9.43	9.66	9.93	10.19								
100	9.33	9.42	9.50	9.67	9.83	10.04	10.25	10.75	11.25	11.75	12.25				
105	9.70	9.78	9.85	10.03	10.21	10.38	10.55	11.03	11.50	11.95	12.40				
110	10.06	10.13	10.20	10.40	10.60	10.73	10.85	11.30	11.75	12.15	12.55				
115	10.43	10.49	10.55	10.77	10.98	11.07	11.15	11.58	12.00	12.35	12.70				
120	10.79	10.85	10.90	11.14	11.37	11.41	11.45	11.85	12.25	12.55	12.85				
125	11.16	11.21	11.25	11.50	11.75	11.75	11.75	12.13	12.50	12.75	13.00	13.50	14.00	14.50	15.00
130			11.65	11.88	12.10	12.12	12.13	12.47	12.80	13.09	13.37	13.80	14.22	14.69	15.15
135			12.05	12.25	12.45	12.48	12.51	12.81	13.10	13.42	13.73	14.08	14.43	14.87	15.30
140			12.45	12.63	12.80	12.85	12.90	13.15	13.40	13.75	14.10	14.38	14.65	15.05	15.45
145			12.85	13.00	13.15	13.22	13.2	13.49	13.70	14.08	14.46	14.66	14.86	15.23	15.60
150			13.25	13.38	13.50	13.58	13.66	13.83	14.00	14.42	14.83	14.96	15.08	15.42	15.75
155			13.58	13.73	13.87	13.97	14.06	14.18	14.30	14.68	15.06	15.19	15.31	15.61	15.91
160			13.92	14.08	14.23	14.35	14.46	14.53	14.60	14.95	15.30	15.43	15.55	15.81	16.07
165			14.25	14.43	14.60	14.73	14.86	14.88	14.90	15.22	15.53	15.66	15.78	16.00	16.22
170			14.59	14.78	14.96	15.11	15.26	15.23	15.20	15.49	15.77	15.90	16.02	16.20	16.38
175			14.92	15.13	15.33	15.50	15.66	15.58	15.50	15.75	16.00	16.13	16.25	16.40	16.54
180									15.80	16.05	16.30	16.40	16.50	16.65	16.80
185									16.10	16.35	16.60	16.68	16.75	16.91	17.06
190									16.40	16.65	16.90	16.95	17.00	17.16	17.31
195									16.70	16.95	17.20	17.23	17.25	17.41	17.57
200									17.00	17.25	17.50	17.50	17.50	17.67	17.83
205													17.80	17.98	18.16
210													18.10	18.30	18.50
215													18.40	18.62	18.83
220													18.70	18.94	19.17
225													19.00	19.25	19.50

Reinforced-Concrete Ties after Service

After several years' service in main-line track on the Pennsylvania R.R. the Riegler type of steel-incased reinforced-concrete tie has been reported on by the Tie Committee of the American Electric Railway Association thus:

RIEGLER CONCRETE AND STEEL TIES ON PENNSYLVANIA TRACK

All the ties are still in good condition and giving satisfactory service, with no apparent depreciation (after 6½ yr.).

The action of these ties under traffic shows that there is considerably less deflection of the track than the adjacent wood-tie track. No separate records of cost of maintenance have been kept. The opinion of the supervisor is that much less work is required for both line and surface than with wood ties. An average traffic of about 70 trains per day passes over this track (60 passenger and 10 freight).

It will be remembered that the tie (described in ENGINEERING NEWS, Feb. 17, 1910) has a concrete body, with a curved steel plate on each side, the plate being connected at the top and bottom by transverse riveted straps. The rails rest on flat steel tie-plates and are secured by through bolts and clips, the bolt-heads lying in pockets in the bottom of the tie. Two bent steel rods are embedded in the concrete.

The depth is 7 in. and the width 9⅜ in. on the flat, or 12 in. over the curved sides. The width and form of section are claimed to give such a large bearing that the 15 ties used under one rail length are as efficient as the 18 wood ties per rail used elsewhere in the track; while the curved sides are believed to prevent any tendency of the ties to slew in the ballast. Owing to the stiffness of the tie, further bearing can be given by increasing its length.

The weight is about 800 to 850 lb.; but the inventor says this causes little inconvenience in handling when proper tools are used, while it adds to the stability of the track. The cost of the ties is not given. The annual cost for ties per mile of track, with a uniform supporting area of 80 sq.ft. per rail length, is given as follows in a statement prepared by the inventor: Concrete ties (16 per rail), $875 per mile per year; untreated oak ties or inferior treated ties (20 per rail), $1196 and $1286 respectively. The life of the concrete tie is estimated at 20 or 15 years for roads of medium and heavy traffic, as against 9 or 6 years for wood ties under the same conditions.

Corrosion of "Pure Irons" and Steels Used by U. S. Reclamation Service

Materials used in the construction of semicircular metal flumes are of much importance to the United States Reclamation Service. Their life is determined principally by the rapidity of corrosion. Galvanizing will prevent corrosion for a certain time, but a base metal of superior resistance would be of distinct value. No conclusive evidence has been received which justifies the claim that any one of the special products on the market has any positive superiority over ordinary commercial grades of iron and steel under conditions of use that obtain on our projects.

CHEMICAL ANALYSES OF TEST SHEETS: U. S. R. S.

	"Pure Irons"		Steels	
	No. 1	No. 2	No. 3	No. 4
Silica	0.0716	0.0294	0.0235	0.0317
Sulphur	0.0424	0.0400	0.0490	0.0414
Manganese	0.4330	0.1730	0.8070	0.8640
Phosphorus	0.0296	0.0222	0.1314	0.0352
Carbon (total)	0.1100	0.0826	0.0901	0.1500
Copper	0.1020	0.0505	0.0307	0.2022

LOSSES (PER CENT. OF WEIGHT) ON EXPOSURE; UNITED STATES RECLAMATION SERVICE TEST SHEETS

	Galvanized	Ungalvanized
No. 1 (pure iron)	1.23	4.53
No. 2 (pure iron)	1.41	3.07
No. 3 (steel)	1.32	2.76
No. 4 (steel)	1.44	2.79

RELATIVE LOSS (LOWEST TO HIGHEST); UNITED STATES RECLAMATION SERVICE TEST SHEETS

Project	Galvanized	Ungalvanized
Belle Fourche	No. (1 and 4), 3, 2	No. 3, 4, 2, 1
Carlsbad	No. 2, 1, 3, 4	No. 2, 4, 3, 1
Sun River	No. (2 and 4), 1, 3	No. 4, 3, 2, 1
Uncompahgre	No. (1, 2 and 3), 4	No. 4, 3, 2, 1
Average	No. 1, 3, 2, 4	No. 3, 4, 2, 1

J. Y. Jewett, cement expert, has tabulated the results of observations on 12x24x$\frac{1}{16}$-in. test specimens exposed on four different projects where the destructive action of alkali has been pronounced. All specimens on any one project were exposed the same length of time, but the tests varied from 10 to 13 months on the different projects. The figures are given in the appended tables.—I. C. HARRIS, Engineer in Charge of Inspection of Materials, in the *Reclamation Record*, May, 1915.

A Home-Made Clamshell Excavator of 1877

BY LEONARD GOODDAY*

As bridge engineer of the South Australian Government many years ago I had charge of the erection of a half-mile iron bridge over the Murray River and swamps, at Edwards Crossing, 56 mi. from Adelaide. It was in the bush, and we did not have many uptodate facilities.

FIG. 1. HOME-MADE EXCAVATOR BUCKET DESIGNED AND BUILT BY LEONARD GOODDAY IN 1877

To those who may some day be similarly situated the following should be interesting.

The main bridge was composed of five 120-ft. spans across the river, for which piers were built comprising

*62 Craig, Ottawa, Ont.

two 7-ft. diameter cylinders each. The rest of the bridge was of 60-ft. spans on piers made of two 3½-ft. cylinders each.

For sinking the smaller cylinders some type of special excavator had to be designed. The kind of grab bucket usually made in those days was hexagonal in plan, which gave it somewhat greater spread, but less capacity than was desired. It was therefore decided to build a bucket circular in plan, with only two leaves, so as to have fewer working parts.

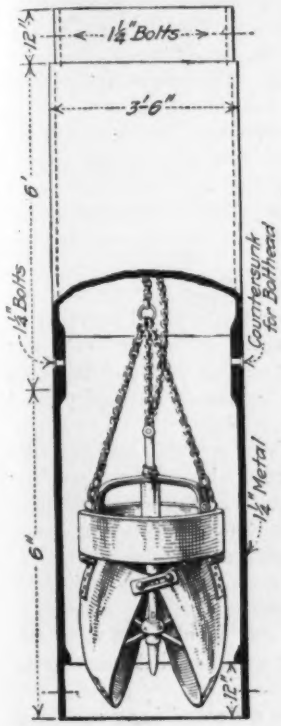


FIG. 2. SKETCH OF EXCAVATOR BUCKET SHOWING CLEARANCE IN FOUNDATION CYLINDER

The cylinders were 3½ ft. in external diameter and 1¼ in. thick, with inside sleeve joints, leaving only 3 ft. 1 in. clear at the joints. The external diameter of the bucket was about 2 ft. 9 in. Fig. 1 is from a photograph of the bucket and Fig. 2 is a sketch showing its clearance in the cylinder.

To obtain proper curves for the leaves several bar-iron templates were made of horizontal and vertical cross-sections, and with these as guides a die was cut in the rock of a nearby quarry. Using the die, ⅜-in. iron plates heated red hot were beaten to shape by hammering.

The links for closing the leaves were ⅜x2-in. bar iron, two to each leaf. The center rod was 2½ in. in diameter, working in a sleeve as shown, with lugs at the lower end for connection with the links. The links were fastened to the leaves by two angle- or bracket-lugs to each link. The upper end of the center rod had a collar stop and a shackle for the chain lift.

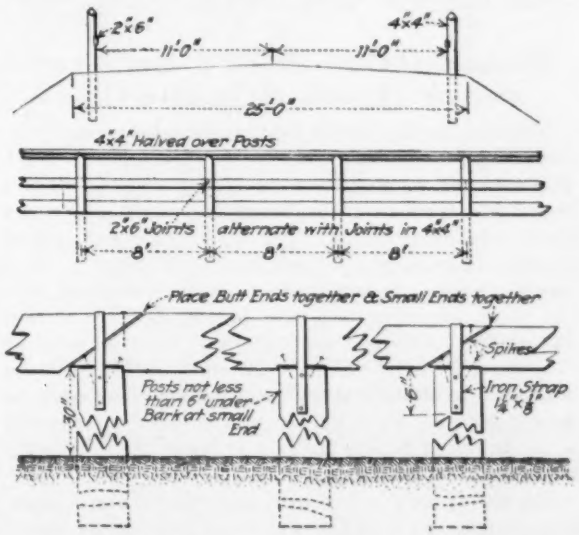
The frame of the bucket was a heavy iron ring, to which the leaves were hinged. Three arms on the top of the ring held the guide sleeve; they were also the points at which chains were attached, forming a three-chain sling by which the bucket was lowered. For additional strength the leaves were shod with forgings as shown.

The amount of material raised at each lift was about 0.44 cu.yd., as against 0.27 cu.yd. if a bucket of hexagonal shape had been used. There were 33 pairs of cylinders sunk at varying depths from 15 to 90 ft. The total material excavated was about 1490 cu.yd., requiring 3387 lifts, as against 5731 which would have been required with a hexagonal bucket, which meant a saving in labor of 40.9 per cent.

Comparative Costs of Guard Rail and Wide Fills on Highways

The general statement is made in a well-known highway engineering handbook that a 4 on 4 slope on the sides of a road fill is cheaper than 1 on 1½ slope with guard-rail protection, until a fill of 7 ft. is reached.

There are, however, a number of elements entering this problem which must be allowed for before a definite



STANDARD GUARD RAILS OF THE MASSACHUSETTS HIGHWAY COMMISSION

decision can be reached. To take account of these varying conditions and yet avoid computation, J. A. Johnston, Division Engineer of the Massachusetts Highway Commission at Springfield, Mass., has prepared the accompanying table and curve.

The table and curve together give the approximate cost of the extra fill (from borrow) required by a 4 on 1 slope, compared with the cost of both standard and

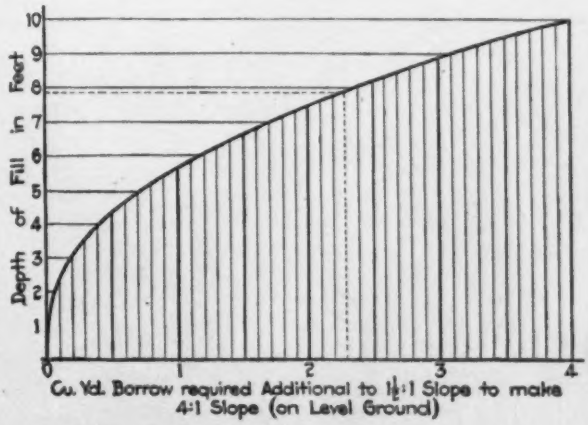


TABLE OF COMPARATIVE COSTS OF GUARD RAIL AND FILL

Cost of Borrow per Cu.Yd.		$0.40	$0.45	$0.50	$0.55	$0.60	$0.65	$0.70	$0.75	$0.80	$0.90	$1.00	$1.10	$1.20	$1.30	$1.40	$1.50
		Cu.Yd.															
Cost of Guard-rail per Lin.Ft.	Rustic	$0.15	1.6	1.4	1.3	1.2	1.1	1.0	0.9	0.9	0.8	0.7	0.7	0.6	0.5	0.5	0.5	0.4
		.20	2.0	1.8	1.6	1.5	1.4	1.3	1.2	1.1	1.0	0.9	0.8	0.7	0.7	0.6	0.6	0.6
		.25	2.5	2.2	2.0	1.8	1.7	1.5	1.4	1.3	1.3	1.1	1.0	0.9	0.8	0.8	0.7	0.7
	Standard	.30	2.9	2.6	2.3	2.1	2.0	1.8	1.7	1.6	1.5	1.3	1.2	1.1	1.0	0.9	0.8	0.8
		.35	3.4	3.0	2.7	2.5	2.2	2.1	1.9	1.8	1.7	1.5	1.4	1.2	1.1	1.0	1.0	0.9
		.40	3.7	3.3	3.0	2.7	2.5	2.3	2.1	2.0	1.9	1.6	1.5	1.3	1.2	1.1	1.1	1.0

*Cu.Yd. of Borrow which can be paid for by the sinking fund necessary to construct, maintain, and reconstruct guard rail (at indicated unit prices).

rustic guard rails (see accompanying illustrations), which are required with a 1½ on 1 slope fill. For example:

Cost of borrow = 50c. per cu.yd.
Cost of guard rail = 30c. per lin.ft.

In the table the number of cubic yards of borrow at 50c., which is equivalent in value to one linear foot of guard rail at 30c., is found to be 2.3. The depth of fill (shoulder to toe) which can be made by 2.3 cu.yd. borrow per lin.ft. is found from the curve as indicated by the dotted line. Depth = 7.8 ft.

Construction Work on Traymore Hotel, Atlantic City

The construction of the new reinforced-concrete Traymore Hotel at Atlantic City, N. J., the design of which was described in ENGINEERING NEWS, July 1, 1915, is noteworthy for the combination of very rapid work and a most complicated design. The old building was vacated on July 1, 1914, and the site cleared by Sept. 1, when construction on the new hotel, a structure covering 140x520 ft. on its lower floor plan and rising to 18 stories, was started. Although much finishing work yet remains to be done, the hotel was opened June 1, 1915, and in the last week in June was sufficiently near completion to house practically all of the 500 who attended the meeting of the American Society for Testing Materials, in addition to the regular guests.

The site is peculiarly disadvantageous for the delivery of materials. It is at the ocean end of a narrow street and immediately on the Boardwalk, which cannot be used for haulage. It was necessary to bring in all material by trucks which had to turn in the dead-end made by the Boardwalk and return along the same street.

The whole concreting was controlled from two plants of similar design located, as shown in Fig. 1, along the street-side of the building. The details of one of the plants is shown also in Fig. 1. It was located so that wagons could drive up on inclined platforms from the street and dump, through screening gratings, into the boo of bucket conveyors which would raise the sand on on side and the stone on the other to bins above. From thes bins gate-controlled hoppers permitted the dropping of th aggregate to the mixer below, whence the mixed concret was dumped into the elevator and carried to the chute t be spouted to its required location. All of the moving parts were driven by an electric motor.

The elevating towers were of very substantial design, approximately 6x8 ft. in plan and 256 ft. high, so that concrete could be spouted to the top of the dome, which is about 230 ft. above the street. The towers were of timber and the height was added to as the building rose, wire guy lines being used to insure stability. For a part of the time each tower was provided with two chuting equipments, as shown in the view in Fig. 2, so that work could be carried on simultaneously at different elevations. The towers also were equipped with 30-ft. booms for handling lumber.

The floors were so large that the spouts could not control the whole area, particularly as the building rose and the spouting angle was reduced; so the concrete was spouted to well located hoppers on each floor and delivered by concrete carts. A small quantity was deposited directly from the spouts.

Most of the concreting was done in winter and as a precaution against freezing, the material bins were heated by steam coils. The water was also heated, and immediately after the centering was completed for the different floors, and before concrete was placed, the floors were entirely inclosed with tarpaulins. Salamanders were placed under the centering and the temperature was kept constantly at 50 deg., until the concrete was about seven days old. Immediately after the concrete was placed it was covered with salt hay.

The concrete was made from a washed bank sand and graded pebbles of maximum ¾-in. diameter, both of exceptionally good quality. Some of the test cubes taken from the concrete when being deposited showed as high as 2800 lb. per sq.in. in compression in seven days and the average of such tests was about 2000 lb. per sq.in.

The foundations, of short timber piles grouped under concrete caps, were very difficult to build on account of the water-bearing nature of the ground, which is beach sand with the water surface only a foot or so below the street level. It was necessary to sheetpile around each footing, or group of footings, and to keep pumps going continuously until the concrete was all deposited. Some of the excavation for these footings was made with a sand pump.

During the first part of the construction both day and night shifts were worked in building the centering, and during that period there were from 100 to 140 carpenters in the day shift and from 60 to 70 laborers helping them. The night gang consisted of from 30 to 40 carpenters and about 20 laborers.

When the job was a little over half done, the local labor unions refused to allow the carpenters to work on night

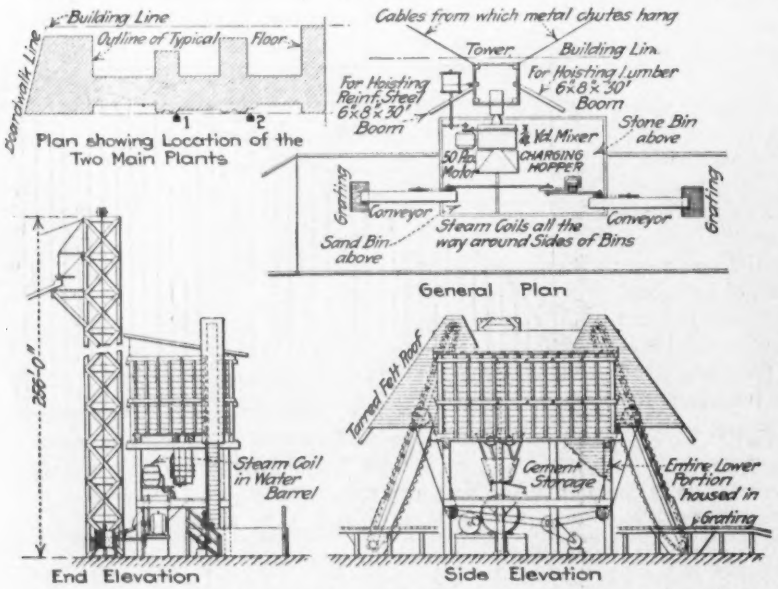


FIG. 1. DETAILS OF CONCRETING PLANT ON NEW TRAYMORE HOTEL ATLANTIC CITY

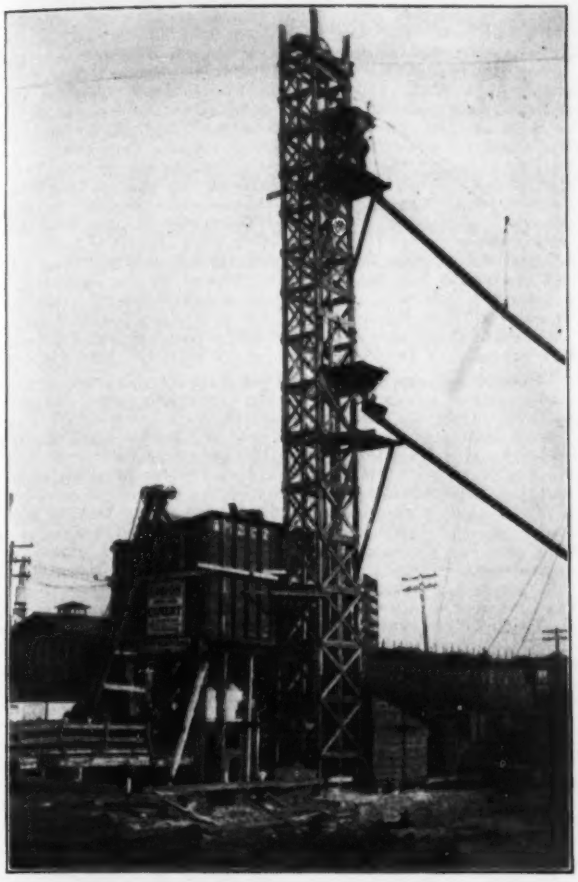


FIG. 2. CONCRETING PLANT AND CHUTING TOWER AT A LOW HEIGHT; TWO CHUTES FROM TOWER

shift. It was possible to increase the regular day shift of carpenters by about 20 men, so that after the night gang was discontinued the day gang consisted of from 150 to 160 carpenters. The concrete was placed either day or night, as fast as the forms were ready to receive it.

With this force and the equipment noted some very speedy work was done. The dates of the completion of the various parts of the concrete work are as follows:

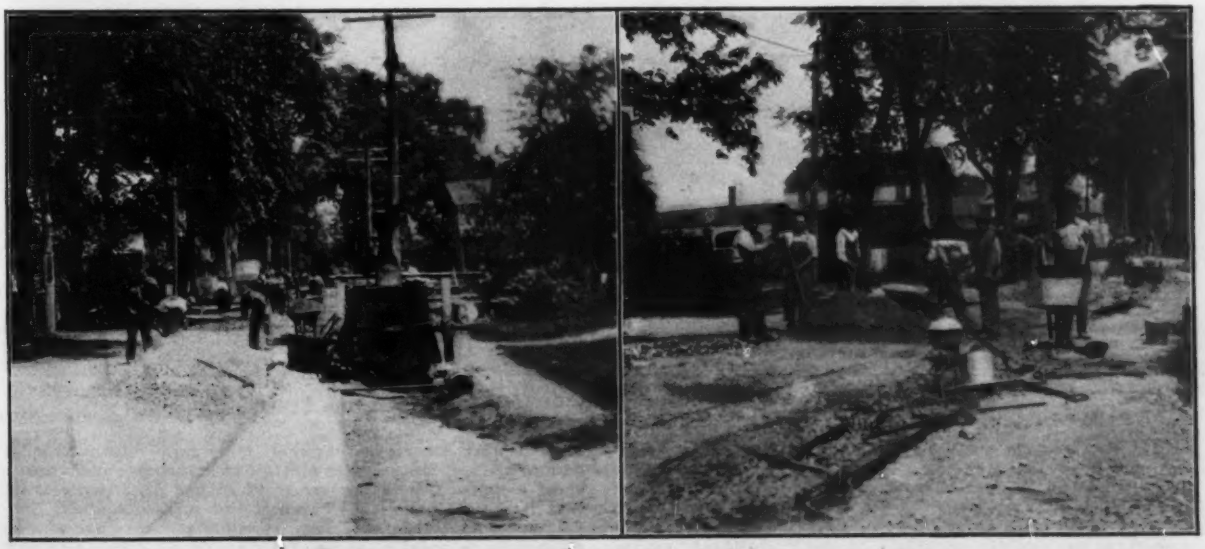
	1914	
Footings	Dec. 27	
Exchange floor	Dec. 30	
	1915	
Mezzanine floor	Jan. 2	Area—140x520 ft.
First floor	Jan. 7	
Second floor	Jan. 12	
Third floor	Jan. 20	
Fourth floor	Jan. 25	
Fifth floor	Jan. 29	
Sixth floor	Feb. 10	
Seventh floor	Feb. 14	Area—about 50x450 ft. with three wings.
Eighth floor	Feb. 20	
Ninth floor	Feb. 28	
Tenth floor	Mar. 5	
Eleventh floor	Mar. 13	
Twelfth floor	Mar. 20	
Thirteenth floor	Mar. 28	
Fourteenth floor	Apr. 2	Area—about 40x150 ft.
Fifteenth floor	Apr. 10	
North Dome	Apr. 22	
South Dome	Apr. 24	

The contract for the building was held by Cramp & Co., of Philadelphia, Penn., and all of the work was done by them under the general direction of F. V. Warren of the company. The architects for the hotel were Price & McLanihan, and the engineers, Otto H. Gentner, Jr., and F. Dickinson Shaw associate engineers, all of Philadelphia.

A Simply Constructed Tar-Macadam Road

In these days of elaborate roadmaking apparatus, complicated specifications, etc., it is occasionally refreshing to see how crudely a successful road can be built. The whole apparatus used in the construction of the Connecticut state-aid road, shown in the accompanying views, were a tar kettle, a 3-ton steam roller, wheelbarrows, rakes, shovels, pails, etc. The mixing was done by hand. With this simple apparatus and a force of a dozen or fifteen men a 3-in. tar-macadam pavement was placed on an old macadam surface for 70c. per sq.yd.—and more than that, the new pavement is guaranteed for five years by a cash deposit.

The contractor for this road is C. L. Farnsworth, Ayer, Mass., who has spent 35 years building tar sidewalks and tar pavements. His pavements are very



A SIMPLY CONSTRUCTED TAR-MACADAM ROAD NEAR HARTFORD, CONN.

similar in construction and looks to the old-fashioned city tar sidewalk, but he has built many miles of them on city and town streets and state roads, and has successfully guaranteed them with cash. Mr. Farnsworth builds these pavements under the trade name of "Ma-Drite." They are not patented; indeed, the contractor frankly informs us that they are nothing else than the "old tar-concrete crosswalks laid throughout New England for 50 or 60 years."

The formula for their construction is as follows:

On the subgrade place 3 in. of No. 1 stone covered with "Ma-Drite" binder, 17 gal. to 1 cu.yd. of stone, rolled until compacted firmly. On this place No. 2 stone (covered with "Ma-Drite" binder, 19 to 20 gal. to 1 cu.yd. of stone) of sufficient thickness to bring the grade 1 in. below the finished grade. These are to be rolled and compacted firmly. On this is to be placed 1¼ in. of pea stone and dust (crusher run) or pea stone and sand, properly proportioned so that the finished surface shall not be slippery. This surface shall be mixed and spread while warm, using 38 to 45 gal. of "Ma-Drite" binder to the cubic yard, rolled and finished in the usual manner.

We find it necessary to vary the quantity of binder owing to the difference in the quality of sand, etc. I always use all the aggregate will hold without the binder coming to the surface and causing a slippery road. I also wish to say that the success of this class of work depends a great deal on the knowledge and skill of the workmen as to the quality of the aggregates and binding material.

The cost of the road illustrated is analyzed by Mr. Farnsworth as follows: Cost of material—stone, $1.63 per ton delivered; sand, $1.15 per cu.yd.; teams, $5 per day; labor, $2.50 per day, with the usual incidentals, such as wood, oils, tools, overhead charges, etc.

This road is a few miles north of Hartford, Conn., and is built under the general supervision of the Connecticut State Highway Commissioner, Charles J. Bennett.

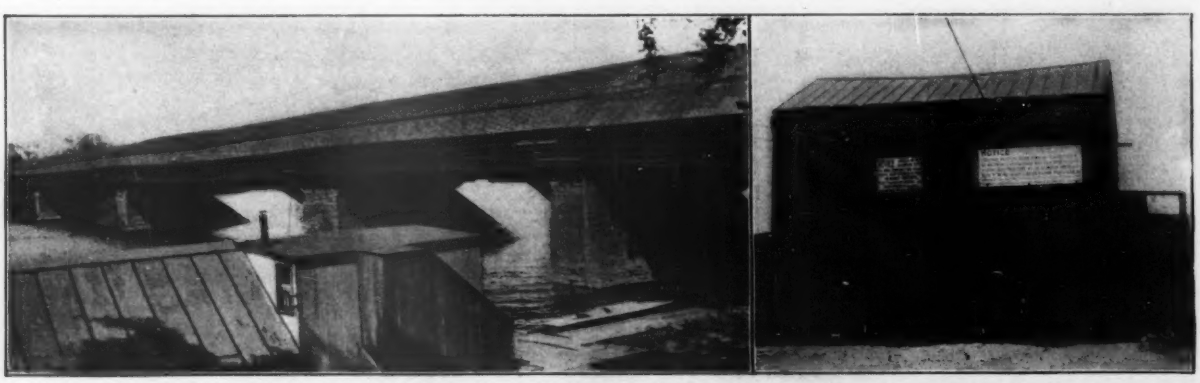
NOTES

A Historic Old Bridge is that shown in the accompanying views. It crosses the Connecticut River at Springfield, Mass., and at the time it was built it was one of the main gateways from New England to the then far west. The original bridge was built in 1804-05. It was financed by a lottery and operated as a toll bridge. The bridge was 1234 ft. long, 30 ft. wide and 40 ft. above low water. There were six spans of 187 ft. each, composed of a framework of timbers, reinforced with wood arches. The roadway, according to an ancient description, followed pretty much the contours of the arches. The superstructure of this bridge was pulled down soon after the freshets of 1814 and rebuilt. This bridge was opened to the public in 1816. In 1817 President Monroe passed over the bridge on his famous tour of the country. It was partly carried away by a freshet in 1818, but was restored in 1820, and has lived and been useful ever since. Tolls were collected until 1872, when the bridge was purchased by the public authorities and made free. The footpath shown in the view was added in 1872 by strengthening the outside truss with a lattice of planks. The bridge proper is made of three Burr trusses, so called. Practically nothing was done to it until 1913, when it became so wobbly that two or three of the spans were strengthened by the addition of some new timber arches paralleling the old and some tension rods. Other places on the interior present curious patches and splices. The old framework was put together with wood pins. It is contrary to law to cross the bridge at a faster pace than a walk. The maximum load is 4 tons, and not more than one heavy load is allowed in any span at the same time. There is a special policeman constantly on duty to enforce these rules. A bill now before the Massachusetts legislature provides for a commission to plan and finance a new bridge.

The Uselessness of General Cost Data—An examination of tables giving the costs of all state highway work in Connecticut during the last two years shows an enormous variation in unit price per square yard of road surface for the same types of road built under the same specifications. The reasons for these discrepancies are many; but principally the variation arises from the location of the road with reference to the source of supply of material used in its surface construction. For instance, many of the roads are located at a long distance from a railroad station or from a supply of native stone or gravel to be used in construction. In addition, the variation is, to some extent, due to the fact that different contractors in need of work bid extremely low prices for road surface in certain cases. Variations in the cost of labor have also been noticeable, and occasionally the material excavated has been used in the surface of the road, thereby reducing the cost.—From the recent biennial Report of the Connecticut State Highway Commissioner, Charles J. Bennett.

The Deepest Boring in the World, according to the United States Geological Survey, is in Upper Silesia, in the German Empire. It is a diamond-drill hole in a coal field and is 7350 ft. deep. A well in the United States which may go deeper is 4 mi. northwest of McDonald, Penn., and about 15 mi. west of Pittsburgh. This well is being sunk to the Medina sandstone, and is now down 7164 ft. Gas and oil were struck in the upper part of the well, and between the depths of 6830 and 7100 ft. rock (bearing rock salt) and salt water were encountered. The temperature in this well at the depth of 6775 ft. was recently determined with accuracy to be 145.8 deg. F. At Derrick City, McKean County, Penn., there is a well 5820 ft. deep, which is probably the second deepest well in the United States. There is another on Slaughter Creek, W. Va., which is 5595 ft. deep. In sinking the last-named well a sandstone was encountered at 5030 to 5050 ft., and from this depth to the bottom, a distance of 545 ft., the well is in limestone.

Steel Roof-Trusses Have Been Destroyed by corrosion in a number of paper mills in northern New York. These frames were erected in the period 1896-1905. In one mill it was recently found that large chunks could be knocked loose from almost any truss member with light blows from a hammer. Yet the steel had retained its shape and from a short distance did not appear to be in bad condition—except that it showed some tuberculation and surface scaling. Examination revealed that every member of one truss was so completely corroded as to make it a puzzling matter to explain how the roof could remain in position.



CONNECTICUT RIVER BRIDGE AT SPRINGFIELD, MASS.; IN SERVICE SINCE 1816

Editorials

Allowance for Transverse Strength in Columns

In discussing the remarkable failure of the Panama Canal crane "Ajax" in the issue of May 13, ENGINEERING NEWS suggested, as one of the lessons of that accident, the propriety of increasing the specification requirement for transverse strength in column latticing to 4% of the column load in place of the 2½% now commonly specified.

Elsewhere in this issue are printed letters from T. D. Monniche, engineer of docks of the Panama R.R.; F. H. Cooke, Civil Engineer U. S. N., author of the description of the "Ajax" wreck published in ENGINEERING NEWS of May 13, and H. F. Welty, bridge engineer of the New York Central R.R. All these engineers dissent from the suggestion as to the increase in specification requirements for transverse strength in column latticing. Mr. Monniche's argument is especially notable because of his intimate knowledge of all the details of the crane's construction. Both Mr. Monniche and Mr. Cooke express the opinion that the failure of the compression member which caused the wreck of the "Ajax" was the result more largely of secondary stresses in the member caused by the connections than of weakness in the latticing. These opinions are, of course, entitled to great weight.

It is well to bear in mind, however, that in no framed structure are the stresses imparted to the members in an absolutely axial direction. In any actual structure there always exist more or less of secondary stresses caused by deformation and by unavoidable inaccuracies in design and in construction. The factor of safety used in proportioning the members is in part designed to make provision for just these variations of stressing, and it was our idea that the secondary effects in this particular crane were no greater than may be expected to occur at any time in any ordinary structural framework.

The so-called shear provision in specifications for columns is meant to cover allowances for torsion, local irregularity, inaccuracies in the application of load, results of distortion, etc. Just how much each of these several elements amounts to in a particular case, we do not try to compute. To do so, indeed, would be impracticable. The best that can be done is to choose empirically a limit which will be safe in practice. Such a limit may indeed provide a large excess of strength in most instances, but it must cover all the cases which practice presents.

The great importance to the engineering profession of the failure of a large member under load is that it furnishes an example which can be calculated to see how our empirical limits correspond to the limit which an actual member passes when it fails.

It may be said that there are two necessary foundations on which to work in reaching a safe load for columns. The first is the ultimate strength which a column exhibits when loaded with greatest care to secure an accurate loading. We can obtain these figures by the use of a testing machine. In the second place we can determine the load at which an actual member fails under the conditions of service, with all the irregularities of load application, secondary stresses, etc., which always occur in practice and use this as a guide for the designer. It is from this point of view that the failure of the compression member in the "Ajax" framework becomes of such importance.

The letter of Mr. Welty raises the question as to what is a reasonable impact allowance for large cranes if 100% is a proper allowance for railway bridges. Mr. Welty states that 50% is used for cranes in the New York Central practice. Possibly on small hoists, which get all sorts of rough and careless handling, such an impact allowance may be justified; but on such a huge crane as that at Panama when heavy loads are being lifted it would be difficult to find an engineering structure to which the load is applied more gradually and carefully.

Unprofitable Street-Car Traffic

In a valuable paper on the rapid-transit problem in cities, by Prof. George F. Swain, published in the April *Proceedings* of the Engineers Society of Western Pennsylvania, considerable emphasis is laid on the unprofitable nature of long-distance traffic on street railways in cities where the universal five-cent fare prevails. Professor Swain states that in making a study of the transportation question in Boston a few years ago it appeared that the limit of a street-railway ride yielding a profit to the company was about 4½ mi. In other words, the cost of carrying a passenger on the Boston street-railway system was in the neighborhood of one cent per mile. Professor Swain notes that this result was only approximate and that similar investigations in other cities under different conditions might show a widely different result and continues:

> **The results obtained in Boston indicate very clearly that it is easy to go too far in supplying facilities for long rides for a single fare. There is no question in the mind of the writer that in most cities of this country long-distance riders, and in fact a large proportion of the passengers, are being carried for less than cost.**

Professor Swain's discussion is along the lines usually followed in dealing with this question; but it seems worth while to raise the question whether this common method of dealing with it does not, after all, involve a fallacy. Is it the distance that makes the carrying of passengers to and from remote suburbs unprofitable? It is true that in the operation of the ordinary surface street railway the short-distance riders who get on the car and off again after having ridden only a few blocks are far more profitable patrons for the company, provided the cars are kept filled, than those who ride half an hour or longer. On the other hand, it is easy to imagine conditions under which passengers could be carried distances of from 5 to 10 mi. at a very small fraction of the one cent per mile which Professor Swain's figures indicate.

The largest item of cost to the railway company in carrying passengers is the wages of motormen and conductors, and these men are paid by the hour and not by the mile. If an express service were operated between a city and an outlying suburb, for example, so that a run of, say, 5 to 8 mi. could be made in 10 or 15 min., the rolling stock and operatives would be used much more effectively in the transport of passengers, and the cost per passenger mile would be correspondingly reduced. A further reduction in the cost would be made if the traffic between the two points was reasonably continuous through the greater part of the day instead of concentrated in the rush hours in the morning and evening.

It would promote clear thinking and accurate statement in dealing with the economics of city-passenger transportation if it were emphasized that the cost to the company of carrying passengers long distances to remote suburbs is due not so much to the distance which they travel as to the fact that they take this journey only in the morning and evening hours. If the cars running from the remote suburbs to the center of the city could be as well filled all day long as they are during the morning and evening rush hours, even these long-distance riders would yield the company a large margin of profit.

It will be admitted by everyone conversant with the street-railway business that what the companies need to do most is to develop short-distance traffic and traffic during the off hours of the day. The first could be brought about by the adoption of the zone system of fares, already in practically universal use in Europe and pretty certain sooner or later to gain favor here. Both these ends might be attained, however, through the sale of passes bearing the photograph of the purchaser and permitting him to ride when and where he pleased on the street-railway system during the time the pass remained valid—a month, a quarter of a year or a year—without further payment of fare.

The holder of such a pass would use it for short-distance riding, when he would walk if payment of fare were required, and he would also use it for travel at hours other than the morning and evening rush hours. Another great advantage of such passes would be the reduction of the delays necessary for the payment of fare when passengers enter the common pay-as-you-enter car. A car earns no revenue while it is standing still, and anything which facilitates rapid movement of passengers in entering or leaving the car is worth an enormous amount to the railway company and is still more valuable to the traveling public in reduction of time necessary for the journey. The general use of such passes would make it possible to eliminate the cumbersome transfer system.

Doubtless the proposal to sell street-railway transportation on such a plan may seem a wildly radical proposal; but there is good precedent for it. Such a system has been in use for years on the Swiss Government railways with most excellent results. It is giving satisfaction to the public and profits to the railways.

It is in line with the most modern ideas with reference to street-traffic problems. A street-railway company expends its money chiefly to provide facilities for handling the traffic. Once these facilities are provided, it is for the benefit of all parties concerned that they shall be used to the fullest extent and produce a maximum of profit combined with a maximum of service.

The Steel-Rail Puzzle

No better indication could be given of the importance of the question, How can good steel rails be secured? than the fact that A. W. Gibbs, Chief Mechanical Engineer of the Pennsylvania R.R., made this the subject of his presidential address at the recent annual convention of the American Society for Testing Materials. That some of the steel rails now rolled are satisfactory is evident from Mr. Gibbs' statement that "if all the rail rolled was as good as the best there would be very little room for complaint." The puzzle is how to frame a specification which will insure that the railway shall receive rails of this satisfactory quality and will exclude rails that are likely to break in service.

In the recent New York Railway Club paper of Gustav Lindenthal[1] and in the letter from a prominent railway engineer printed on p. 35 of the last issue, chief stress was laid on finishing the rails at a lower temperature in order to insure better working of the metal. It has indeed been a general belief among railway engineers for a very long time that rolling the metal too hot is the chief cause of poor rails. At the recent Testing Materials convention, however, one of the subcommittees of the Steel Rails Committee submitted a report on the finishing temperatures of rails. This committee reported tests made by M. H. Wickhorst, the Illinois Steel Co., the Pennsylvania Steel Co. and the Cambria Steel Co. on rails rolled at different temperatures, some of the rails being rolled as cold as was possible without reaching the critical temperature. In all these tests, those conducting them were unable to find any considerable difference between the physical qualities of the rails rolled at low temperatures and of those rolled at high temperatures. The Pennsylvania Steel Co. tested these rails with a machine devised to wear the rail by a revolving car wheel, but no material difference was discernible between the wear of rails rolled at high temperature and of those rolled at low temperature.

The logical result of this committee's investigations would be the abandonment of the shrinkage clause in rail specifications, as was stated in the convention report in the last issue. But engineers naturally hesitate to take the responsibility of introducing any such radical innovation in specifications lest they be charged with omitting some precaution that might conduce to greater safety.

Disappointing though it may be to realize that a radical improvement in the quality of rails cannot be expected by so simple a change as rolling at a lower temperature, it is on the other hand important from the economic view that such a change, which would doubtless result in a material increase in the cost of rails, is not likely to be necessary. As Mr. Gibbs pointed out in his address, the railways have been forced to the use of heavier rolling stock and consequent increase in rail stresses by the steady increase in operating expenses and steady reduction in freight rates. "If the railways had to revert to the old methods, using the type of equipment standard twenty years ago, it is improbable that they could exist under modern conditions and burdens. The sheriff would have them."

Similarly, Mr. Gibbs pointed out, the modern steel mill like the railroad can exist only by being operated on a large scale. Modern steel-rail production is a continuous

[1] "Engineering News," May 27.

process from the time the raw ore and coal are dumped into the blast furnace until the finished steel rail is discharged onto the cooling beds. Any change in method to finish the rails at a lower temperature which would involve an upsetting of this continuous process might result in such an increase in cost as would make the product uncommercial.

Mr. Gibbs was careful not to overstate the situation with reference to rails. The actual percentage of rail breakages is trifling compared with the number in service, and the very great majority of those which break are discovered by vigilant track inspection before any harm ensues. It is because every rail breakage in main line track presents the possibility of a serious accident involving large loss of life that it is worth while to use every endeavor to reduce such breakages. Mr. Gibbs frankly admits, also, that the very heavy wheel loads and high train speeds, with imperfect conditions of track maintenance, are responsible for much of the rail breakage. There is not likely to be a material reduction, however, in either loads or speeds, and the means available for maintaining track are too crude to expect perfection or anything approaching it.

Probably the most important move toward a better quality of steel in rails is the very general change to openhearth instead of bessemer steel. The railways have been willing to pay an increased price for openhearth rails in order to secure a safer metal. The willingness of railways to spend money in order to obtain greater safety is illustrated by Mr. Gibbs' statement that on one railway system the actual expense of every kind caused by rail breakage in a year was less than one-fifth the amount of the increased cost of rails because of the introduction of a proposed new specification designed to secure better quality rails.

As to the possible methods by which the quality of the metal in rails may be improved, Mr. Gibbs names three which are now being tried to the extent that rails produced by them are in service in the track. These are the electric furnace, the sink head process for casting ingots and the Lackawanna Steel Co.'s deseaming process.[2] Heat treatment of rails is also being tried and Mr. Gibbs thinks it is promising. His personal opinion is that the time has arrived for a considerable increase in the weight of rail sections and that this rather than improvement in quality of the metal is the most promising means for reducing rail breakages at the present time. He would have no halt made, however, in the search for a means to produce rails of more reliable quality, and particularly in the study of methods which will enable the engineer to be more certain as to the quality of the rails that he purchases.

At present neither the standard rail specification of the American Society of Testing Materials nor that of the American Railway Engineering Association really protects the buyer as to the chemistry of the rail which he purchases. It has been demonstrated that the chemical analysis of the finished rail often varies very widely from the analysis of the specimen taken from the ladle before pouring the ingot. It may yet be necessary in rail specifications to do as has been done in some of the specifications framed by Dr. Dudley for the Pennsylvania R.R.—make the method of chemical determination a part of the specification.

[2] "Engineering News," Dec. 17, 1914, p. 1228.

Unemployment Among English Engineers

It is doubtless little realized in this country how serious is the situation that confronts the civil-engineering profession of Great Britain. In an editorial entitled "Employment After the War," London *Engineering* reveals the extent to which the abandonment of ordinary construction work has made idle many hundreds of engineers. In Great Britain at present everything is subordinated to the one task of vigorous prosecution of the war, especially to securing an ample supply of guns and munitions for the British troops and their allies. This means the suspension of nearly all civil-engineering work, mainly because funds for it are lacking. All the money which can be secured from investors is devoted to the prosecution of the war. Cities are required by the Local Government Board to make only such extensions and improvements to their water-supply, sewerage and drainage systems as are absolutely essential, and such work is limited to what can be paid for by taxation.

British cities must not compete with the government in the market for capital. A similar condition exists with respect to county work; the railways are all in the hands of the government and private firms are not undertaking extensions of plant or buildings. Even if capital were available to undertake engineering works, the labor is lacking to carry them on.

Under these conditions the civil engineers of Great Britain, unless they happen to be connected with some of the establishments which are producing ammunition or necessary supplies for the war, find their occupation gone. They cannot mend their situation, either, by emigrating to the colonies, for the tying up of capital by the war has put a stop to constructive enterprise there as well, and the same thing is true of such regions as South America and China, which have been accustomed to draw from Europe the capital required for their engineering works.

Engineering points out, however, that as soon as the war is over a great demand will arise for the immediate construction of work long deferred. For one thing, such work will be necessary in order to provide employment for the hundreds of thousands of soldiers returning to civil occupations. Our contemporary urges, therefore, that the engineers now idle should be set at work making surveys, plans and specifications for works to be undertaken when the war shall end, so that when the emergency comes, the plans for such work will not have to be made in a rush, with insufficient investigation.

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An amendment recently introduced in the New York Constitutional Convention reads as follows:

> **The legislature shall not pass any bill nor shall a state board, commission or officer adopt any rule or regulation affecting manufacturing industries or business which is unreasonable.**

If only it were possible by constitutional amendment to so restrict our legislators and public officials that they should subject no one, high or low, to unreasonable laws or restrictions! If the New York Constitutional Convention could work that miracle for the benefit of the people of the state it would confer such a boon upon them that it could afford to forget all the other measures before it. But who would decide where reason ended and unreason began?

Letters to the Editor

Stresses in the Panama Canal Cranes

Sir—I have just read, on p. 947 of ENGINEERING NEWS, May 13, 1915, your comments on the article written by Civil Engineer F. H. Cooke, U. S. N., on the failure of the Panama Canal crane "Ajax." I note your statement: "The crane is a type of engineering structure in which it is possible to compute the stresses with greater accuracy and certainty than in almost any other structure which the engineer builds," and also the conclusion, "The safe course is to make latticing strong enough for 4 per cent. of the column load."

In view of the great influence such a conclusion, appearing in one of the foremost American technical journals, will have on the work of the structural engineer, I desire to present my views on the matter, particularly as the plans for the strengthening of members 27, 28 and 29 were prepared under my immediate direction, and the strengthening of the crane "Hercules" was executed in accordance therewith while Mr. Cooke was absent on leave.

Referring to your statement regarding the exact computation of the stresses in all the members of the crane, your attention is called to the fact that no spherical pins have been provided at the intersection of members 28 and 29, whereas such pins have been used at the heel of the jib. The reason for using spherical pins at the latter place is that it is a practical impossibility so to align these pins as to avoid excessive bending and torsional stresses in the adjacent members, as well as increased friction in the journals while revolving the jib.

The crunching sound mentioned by Mr. Cooke in connection with the "Ajax," and which was also heard by the writer during the tests of the "Hercules" after it had been strengthened, clearly demonstrates that the heel pins on both cranes are out of alignment. The perfect alignment of the pins at the intersection of members 28 and 29 is even more difficult to obtain than the alignment of the heel pins. It is therefore fair to assume that the pins at the intersection of members 28 and 29 are not in proper alignment. If this be the case, and assuming members 28 and 29 to be perfectly rigid against torsion and bending, the pin at the intersection of these members will describe a warped surface of a definite geometric shape when the jib revolves, whereby the angular inclination of the pin relative to the longitudinal and transverse axes of the connecting link will continually change, causing torsional and bending stresses in the connecting link. As, however, none of these members are perfectly rigid, they will all distort due to torsion and bending, and the consequent inclination of the axis of the pin will change from its geometric position under the previous assumption to one dependent upon the combined influence of the strength of each member to resist torsion and bending, the angular deflection of the pin caused by each member being in inverse proportion to its strength. The analysis of this problem involves complicated calculations, which will not give results with the accuracy and certainty stated in your comment.

A general idea of the magnitude of the various stresses produced in the members by the improper alignment of the pin may be obtained from the following discussion:

Members 28 and 29 are fastened together nearly at right angles to each other by two gusset plates. Any bending or torsion in member 29 produces torsion or bending respectively in member 28, and vice versa. As these members are adapted to resist bending stresses better than torsional ones, any change in the inclination of the pin with reference to any plane is resisted principally by the strength of these members in bending. On the other hand, the connecting link is so constructed that it is not well adapted to resist either torsion or bending; therefore the inclination of the pin is principally dependent upon the bending strength of members 28 and 29. This will result in a condition of loading approximately the same as that previously described, in which members 28 and 29 were assumed perfectly rigid and all distortion due to torsion and bending was taken up by the connecting link. In analyzing the strength of the connecting link it can readily be seen that this member may, without serious weakening, be subjected to torsion, but it cannot be subjected to great bending, since this member consists only of two large plates fastened together by small batten plates spaced about four feet apart. Since this member cannot resist great bending, the combined bending and tension in this member will result in the unequal distribution of tension between the two sides of the link, which by increased angular inclination of the pin may result in the total load being carried by one side of the link, thereby subjecting member 29 to nearly its full load applied at one side, thus producing excessive bending stresses, resulting in probable failure.

Referring to the detail construction of member 29, at the heel of the jib, it will be seen that the unsupported length of gusset plate was originally about 6 ft., no diaphragms having been provided at this point for member 29, the only connection between the two plates being three small batten plates in No. 28, each 9 in. long. At the other end of member 29 the unsupported length of gusset plate was also about 6 ft. It is evident that, when a column of this character is subjected to eccentric loading, buckling will take place in the gusset plates if no provision is made for transferring the shear produced by the bending. It is therefore believed that, as far as member 29 alone is concerned, failure was due to the omission of diaphragms between the gusset plates at each end, and that the lacing bars and batten plates were sheared off after the buckling of the gusset plates took place. Inspection of the lacing bars and batten plates throughout the structure will show that they are weaker than is usual in American practice; and although the strength of the lacing bars for member 29 is comparatively less than for any other member of the structure, it should be noted that this member was

furnished with plate separators connecting the webs of each side of the member, which tend to increase the strength of this member for centric loads.

The lower chord of the jib, which is about 120 ft. long, is furnished for its total length with only two small batten plates, one at each end, whereas American practice would call for one batten plate at each end of each panel, or, in other words, for a total of 14. This member showed no weakness; on the contrary, as can be seen from the photographs, it withstood wonderfully the shocks to which it was subjected when the jib collapsed. In view hereof it is not believed that the present American practice regarding lacing bars and batten plates for compression members is unsafe, nor is it believed that the failure of member 29 was caused by insufficient strength of lacing bars or batten plates.

The principal strengthening of member 29 was the provision of diaphragms at each end to prevent buckling of the weakest part—the gusset plates. As the detail of the heel and of the strut would not permit the placing of the diaphragms as desired, a yoke of plates and angles was added on the outside of the strut as further precaution against buckling. Heavier lacing bars, with stronger rivet connections, were also supplied, but solely for the purpose of taking care of the increased bending stresses that might be produced in the strut by the eccentric loading which, as pointed out in the foregoing discussion, is likely to occur on account of improper alignment of the pins.

The upper end of strut 29 is braced against lateral motion by a portal at member 28, a sway-frame at member 23, and lateral bracing at member 25. As member 28 consisted of two built-up channel sections connected by double lacing of one-rivet connections, the upper part of member 29 was braced against lateral motion by the value of one rivet only. In order to increase this value, the double lacing bars were replaced by a solid web. As far as member 27 is concerned, the inclination of the lacing bars was found to exceed that of common American practice, and since it was found impracticable to change this inclination, batten plates were substituted.

Regarding the general construction of the jib, it should be noted that the top lateral system has intentionally been omitted in order to allow the two vertical trusses so to adjust themselves in their own planes that both trusses will at all times carry equal loads. This method of construction would work admirably in case a sway-frame could be placed at member 29. As the details of construction are such that a sway-frame cannot be placed at member 29, and as it is essential to the construction of the jib to have it stayed at the intersection of members 28 and 29, this was accomplished by the portal, sway-frame, and bottom lateral system previously mentioned. By this method of construction, however, the great advantage of omitting the top lateral system is offset by the fact that any adjustment of the vertical trusses to equalize the loads they are to carry causes lateral deflections at the intersection of members 28 and 29, which in turn introduce bending in member 29, the calculation of which is equally as complicated and uncertain as that due to improper alignment of the pins.

In conclusion, I wish to state that, in my opinion, the primary cause of the failure of the jib was the excessive bending stresses produced in member 29 by either an improper alignment of the pins or by the lateral deflection produced by the portal, or by both. The secondary cause was the omission of diaphragms at each end of member 29. In the event that these had been provided, that the pins were properly aligned, and that the portal caused no deflection, I see no reason why failure should occur in either the lacing bars or batten plates.

The plans for stiffening the various members as described in this letter show what was done in the effort to make the jib already erected on the "Hercules" as strong as possible without dismantling and re-erecting. I wish, however, to state that before the "Hercules" was tested I gave no assurance that the method of reinforcing would be successful, as it was not as good as could have been employed if the jib were rebuilt entirely.

As the writer has not yet had the opportunity to make the required calculations, it has not yet been definitely established whether the successful tests of the "Hercules" were due solely to the stronger members, to the more perfect alignment of the pins, or to both.

T. B. MONNICHE,
Engineer of Docks, Panama R.R.

Cristobal, C. Z., May 25, 1915.

[Comment on the foregoing letter is made in our editorial pages.—EDITOR.]

Sir—I note that in the editorial comment on my account of the failure of the floating crane "Ajax" the conclusion is reached that latticing of compression members should be capable of resisting a transverse shear equal to 4 per cent. of the calculated column load instead of the 2 or 2½ per cent. at present considered sufficient. I think that the statements made in the article hardly justify such a conclusion.

It is true that the principal or primary stresses in structures like the crane "Ajax" are susceptible of more satisfactory computation than corresponding stresses for railway bridges, for the reason that it is not necessary to make more or less arbitrary assumptions for impact and vibration, but it is very possible that secondary stresses not susceptible of such satisfactory computation may exist, and may have an important bearing on the resultant strength.

In the next to the last paragraph of your editorial you draw attention to the unusual character of the strut, particularly as regards the detail at its upper end. These conditions were brought out in the description of the general construction of the jib contained in the article. I was absent from the Isthmus at the time of the failure and did not return until about two months thereafter. T. B. Monniche, Engineer of Docks, made an investigation of the conditions developed by the failure, and I understand that he is of the opinion that the primary cause of the failure was the absence of correct alignment of the pin at the intersection of members 28 and 29. If this pin was out of line, it might well produce secondary stresses of considerable magnitude, though difficult to compute satisfactorily, and the effect of this misalignment would be felt in compression member 29. This member was not well designed to resist forces producing shear between its individually weak halves; although it would in all probability have withstood with ease an axial compressive force of the magnitude of the theoretical compression existing in it at the time of the failure.

Therefore, I am of the opinion that a conclusion that the latticing of all columns should be strong enough to resist a transverse shear equal to 4 per cent. of the column load does not logically follow from the "Ajax's" failure, for there are too many elements of doubt to justify such a conclusion.

F. H. COOKE,
Designing Engineer.

Balboa Heights, May 29, 1915.

Sir—Referring to the description of the failure of the Panama Canal crane "Ajax" and your editorial comments on this failure:

The formula for transverse shear to be taken by lacing bars, which appears in the New York Central Lines Specifications, is based on the cross-sectional area of the member, assuming that the member has been designed to meet the specification requirement as to the loads and unit stresses. This formula would give, in the case of the "Ajax" crane, a transverse shear of about 2½ per cent. of the calculated axial stress.

In the writer's opinion your conclusion that this requirement should be increased to 4 per cent. is not warranted by the failure under discussion.

The calculated unit stress in the heel strut under conditions which produced failure—that is, a dead-load plus a live-load of 580,000 lb.—is practically equal to the stress permitted by the Canal commission's specifications; but apparently no allowance has been made for so-called "impact," and it seems entirely probable that the heel strut was subjected to a stress considerably in excess of the calculated stress. There was a considerable inertia to be overcome in moving the load of 135 tons when the luffing pull was applied. Thus for an instant after the luffing motors were started the stress in the heel strut was probably considerably in excess of the stress due to dead- and live-loads alone.

Besides being increased by the inertia of the load, the calculated stress might have been also increased by an unequal distribution of the load to the two trusses.

The failure of a column at a unit stress of 12,000 lb., even with poor details, seems unreasonable; but assuming that this stress was increased 50 per cent., and considering the poor details and the fact that the member was unbraced, the result was not extraordinary.

In the writer's office an impact allowance of 50 per cent. is added to the lifted load when calculating stresses in crane girders. Assuming that lacing bars should have been designed for a shear equal to 4 per cent. of the stress due to dead-load plus live-load, we would get a shear transverse to the axis of column of 580,000 × 0.04 = 23,200 lb. Assuming again that an impact allowance of 50 per cent. should have been added to the lifted load, the designing stress in the strut would have been 147,800 lb. + 432,300 lb. + 216,150 lb. = 796,250 lb.

The shear of 23,200 lb. (4 per cent. of the dead-load and live-load stresses combined) is practically 2.9 per cent. of the stress, which includes 50 per cent. impact, and agrees closely with the New York Central Lines' specification requirements, considering the fact that these specifications are for railroad bridges and provide for a maximum impact of 100 per cent.

The stress in the lacing bars is determined by multiplying this shear by the secant of the angle which the bars make with a line normal to the axis of the column. In this case the bars were inclined at an angle of practically 45 deg., although first-class practice would require that they be placed at an angle not greater than 30 deg., with effective stay-plates at the ends of the member. The rivet shear at any lacing-bar connection would be equal to the sum of the longitudinal components of the stresses in the two bars meeting at that connection.

An analysis of the rivet stress in the lacing-bar connections of the member under discussion, assuming an increase in stress for lifted load of 50 per cent. and a transverse shear equal to 2½ per cent. of the axial stress—which in this case would conform with the requirements of the New York Central Lines' specifications—shows the following result:

Transverse shear on two planes = 0.025 × 796,250 = 19,900 lb.

$$\text{Shear on one } \tfrac{3}{4}\text{-in. rivet} = \frac{19{,}900 \times 2}{2} = 19{,}900 \text{ lb.}$$

$$\text{Shear per square inch} = \frac{19{,}900}{0.44} = 45{,}200 \text{ lb.}$$

At an allowable unit stress in shear of 13,500 lb. per sq.in., one ¾-in. rivet would have a value of 5940 lb., so that the lacing-bar stress of 14,000 lb. would have called for a three-rivet connection.

While the foregoing, in the writer's opinion, justifies the use of the formula for lacing bars contained in the New York Central Lines' specifications, it is not meant to justify the use of lacing bars in such a case as that under discussion. In this connection your attention is called to the following paragraphs in that specification.

Par. 66. . . . Compression members shall have their segments connected by solid webs, if practicable.

Par. 67. The open sides of compression members without central webs transverse to the plane of the segments shall be stayed by stay-plates and lacing bars. The open sides of the compression members with such central webs shall be stayed by stay-plates. The stay-plates at the ends of members shall be placed as near the end of the member as practicable, and shall have a length not less than one and one-fourth times the width of the member between the nearest rivet lines connecting them to the flanges.

H. T. WELTY,
Engineer of Structures, Exterior Zone,
The N. Y. C. R.R. Co. East.

Grand Central Terminal,
New York, June 4, 1915.

Competitions as an Aid to Engineering Instruction

Sir—Recently instructors in two of the leading technical schools of this country have commented on a marked difference between the students of the departments of civil and architectural engineering as regards their attitude toward their studies, capacity for work, enthusiasm and *esprit de corps.* The writer, also, from experience as a student and a teacher in both architectural and civil engineering, believes that this distinction exists and is not due to dissimilarity of character and qualifications of the students in the respective courses but to essential differences in the administrative and pedagogical methods of the two departments.

The courses in architectural engineering are so taught that the student first learns the fundamental principles and then their application to the solution of a number of practical problems. Each student solves the same problem—but independently, in an original manner and with-

in a limited period of time. In many of the more advanced courses the students' work is graded and ranked in order of merit by a group of judges composed of members of the instructional staff and often one or more distinguished practitioners. The results for each problem are publicly exhibited and at a stated time are reviewed and criticized by the instructor in charge. This method of competitive and coöperative endeavor on the part of the students arouses in them a healthy spirit of emulation and enthusiasm. The solution of each problem becomes an interesting game in which all participate with that earnestness and industry which are essential elements for success in their school training as well as in later life.

It is true that the courses in civil engineering also require the solution by the students of a large number of practical problems; but the writer does not know of a single school where these problems are solved on a competitive basis. The usual abstract and noncompetitive methods of field, laboratory and drafting-room work lead to individual, isolated and an unenthusiastic effort. The most earnest and enthusiastic teaching cannot inspire the student with the same interest in his work as does the knowledge that he and his work are constantly up for judgment before the whole school. Awaken a student's pride and you will arouse him to high endeavor, while the mere acquisition of "passing grades" will hardly more than maintain his flagging interest.

It is believed that the competitive method of instruction could be successfully applied to many courses in still other branches of engineering. In order to become effective, this method would have to be adopted as a department policy and all the members of the instructional staff and student body would have to coöperate. It would be desirable in some of the more advanced courses to secure the coöperation of agencies outside of the school, such as engineering companies, technical societies and public organizations, in the formulation of problems, criticism of the results and the possible award of tokens of merit.

A. B. McDaniel,
Assistant Professor of Civil Engineering,
University of Illinois.

June 9, 1915.

Making the Most of the Working Season on Public Works

Sir—Much municipal work suffers from the lack of system in preliminary details. For example, there is the custom of beginning in the spring to arrange for paving, dragging the necessary legislation along for weeks, letting the contract in midsummer or later, and beginning actual construction so late in the season that bad and perhaps freezing weather in the fall overtakes the contractor. This almost always results in added expense to the contractor and injury to the concrete and to the cement-grout filler, if a brick pavement is being put down. Much has been said and written about greater efficiency in construction methods employed on public works, nor has too much emphasis been placed on this phase of the subject; but there is just as urgent need for system and efficiency in preliminary proceedings.

The present illogical system, or lack of system, is chargeable partly to what seems to be the natural tendency of public officials to dilatory action and partly to the laws which govern political units. So far as public officials are concerned, aroused public sentiment could be made to act as a spur; but sometimes laws enacted by state legislatures are so firmly rooted in custom that it is difficult to get them changed.

In most states all municipal and county officials are inducted into office in the early part of the year. Time is consumed in organizing the government, familiarizing the personnel with their duties, and in formulating plans and devising means for putting them into effect. Ordinarily the budget of receipts and appropriations for the year is made out in the spring. Usually a city administration which is to go out in the spring does not assume the responsibility of authorizing expenditures for work to be done under the succeeding administration.

Some change ought to be effected in these matters, either by legislative enactment or through an iconoclastic assault by public opinion upon the idol of procrastination so long worshipped by public officials, so that all preparatory work shall be got out of the way during the winter months and construction work begun as early in the spring as the weather permits.

F. A. Churchill.

Conneaut, Ohio, June 22, 1915.

Selling Water by Meter

Sir—The editorial comment on my letter in your issue of June 3 concerning water meters should in fairness permit a reply.

The policy of charging for public use, and for facilities provided though not used, will undoubtedly reduce the cost to actual consumers; but these matters are obviously not related to the economic question involved in metering small consumers under conditions similar to those existing at Portland, Ore.

As to selling water at flat rate based on estimated legitimate consumption, which you apparently condemn as unfair and absurd under all circumstances, cases can be cited in which actual experience has shown that the general use of meters resulted in increased cost or decreased profits. Two such are personally known to me, one a hydro-electric plant and the other a privately owned water company which pumped all water against a 240-ft. head. In both cases the books showed that the use of meters on small consumers resulted in financial loss.

In Portland the fact in regard to distributing mains is that in small sections, through poor planning and the purchase of inadequate private mains, the present legitimate consumption unduly lowers the pressure at summits, and therefore it is impossible that meters will remedy this trouble except by objectionably curtailing the legitimate use.

The high peak of the consumption curve due to general lawn sprinkling for a few hours in the early evening is the direct cause of the insufficient pressure in limited districts, and this can be overcome for the present mains only by regulations distributing the hours of sprinkling.

In any case, it appears axiomatic that only if the value of the water saved, or the value of the extended term of useful service of existing mains, equals or exceeds the total costs of metering is the metering of all consumers justified as a business proposition.

George Rae.

34 Jefferson Ave., Brooklyn, N. Y., June 12, 1915.

News of the Engineering World

Convention of American Institute of Electrical Engineers

The thirty-second annual convention of the American Institute of Electrical Engineers was held at Deer Park, Md., June 29 to July 2. There was the usual varied program of papers and entertainment. The attendance (200) was good in spite of possible interference with the special September convention in connection with the International Engineering Congress at San Francisco.

While parts of the technical program were of greatest interest to various electrical specialists, yet a considerable number of papers were of general engineering interest and are briefly reviewed here.

Power Transmission

A series of short papers, by J. A. Walls, of the Pennsylvania Water & Power Co., W. E. Mitchell, of the Alabama Power Co., P. M. Downing, of the Pacific Gas & Electric Co., F. C. Connery, of the Toronto Power Co., and J. B. Leeper, of the American Bridge Co., described tower footings that were unsatisfactory and others that amply provided against uplift, compression and horizontal movement. Methods of tower erection were briefly noted.

A paper by A. Bang, "Four Years' Operating Experience on a High-Tension Line," gave information about the Holtwood-Baltimore 70,000-volt system of the Pennsylvania Water & Power Co. and described the failure of about 3 per cent. of the porcelain insulators—due possibly to unburned spots or cracks caused by thermal and other service stresses. Special devices were described which had reduced the number and severity of disturbances caused by lightning; these involved (1) shunting the short-circuit arc momentarily with a fuse across the line in trouble and (2) momentarily reducing the generator field excitation.

Electric Railroads

A series of papers were presented showing the construction and maintenance methods and costs of contact systems on various heavy electric roads (all of which have been described at various times in Engineering News).

E. J. Amberg described the catenary lines of the New Haven, New York, Westchester & Boston and Boston & Maine (Hoosac Tunnel) lines, quoting the construction costs as running from $5000 (tangent single catenary) to $12,500 (4-deg. compound catenary) per mile. F. Zogbaum described the organization of maintenance on the New York, Westchester & Boston, and gave various data on failures and maintenance costs, the total average monthly cost for the last half of 1914 being $17.61 per track mile of contact system and $2.69 per mile of feeders. The total of this electrical maintenance for December, 1914, was 1.42c. per car-mile.

J. B. Cox described the 1500-volt catenary trolley system and locomotive collectors (roller pantographs) of the Butte, Anaconda & Pacific Ry., and gave the line-construction cost as $5514 per mile. The troubles overcome in operation were pictured, particularly those with the collectors. The maintenance cost for nine months to April, 1915, was at the rate of $95 per track mile per year.

J. V. B. Duer outlined the protected third-rail construction of the West Jersey & Seashore R.R. (Pennsylvania Lines) which had cost $4235 per mile for the third-rail system and $679 for the track bonding. The principal trouble described was that due to sleet. The detention figures were given—497 min. per year due to the third-rail system. The maintenance cost was $82 per mile per year (average of seven years) for the third rail and $26 for the track bonding. The much higher figures of the discarded trolley wire were given for comparison.

Experience with the unprotected 600-volt conductor rail of the Chicago Elevated Ry. was presented by C. H. Jones, showing a first cost of $3284 per mile and a maintenance burden of $80 per mile per year.

The design and materials for the 1550-volt overhead lines of the Portland Division of the Southern Pacific Co. were given in considerable detail by Paul Lebenbaum, with tabulated costs of material and labor. Briefly summarized these showed a first cost of $2550 per mile of track. The maintenance burden was $78 per mile per year.

C. J. Hixson, of the General Electric Co., presented curves showing the variation of factors affecting the design of proper contact systems, especially catenary trolleys.

Street Lighting

At a joint session with the Illuminating Engineering Society, P. S. Millar, of New York City, presented a lengthy paper, "Effective Illumination of Streets," and accompanied it with demonstrations by small lamps on a miniature street. He classified various streets and described the varied aims in illuminating them. Particular attention was given to vehicle driving. It was emphasized that objects were discerned usually more by contrast perceptions than by actual illumination. The important rôle of silhouetting against specular reflection on polished pavements was dwelt on. Glare was to be reduced by more lamps and lighter surroundings rather than by any shading which prevented specular reflection. The necessity was shown of arranging road lamps so that their glare did not obscure important stretches behind them.

Use of Motors

A discussion of the engineering factors which have to be studied in adapting electric power to various industries was opened by D. B. Rushmore, of Schenectady, for the Industrial Power Committee, with a general analysis of manufacturing industries, processes, equipment, labor and cost factors. This was followed by brief reviews by several members of the power requirements and service difficulties of elevators, rubber works, printing shops, brick, cement,

pulp and sugar mills, and of excavating and dredging operations in the field.

Electric Cooking

Definite data on the operation of several typical commercial electric ovens were offered by P. W. Gumaer, of St. Louis, in the paper "Economic Operation of Electric Ovens." The objects of cooking were reviewed as aimed (1) to increase digestibility of food, (2) to make it more palatable, or (3) to sterilize it. The most economical temperatures and lengths of time for oven cooking of various foods were found by a long series of trials. This necessitated finding the energy lost in preheating an oven, in maintaining it at constant temperature, and in opening the doors. The most economical temperature for well-done beef roasts was 120 deg. C.; for rarer conditions 100 deg. There was a great saving by searing the roast on top of the stove instead of preheating the oven to 250 deg. for this purpose.

Biscuits were best baked in temperatures of 200 to 240 deg. C. The energy required was the same for any temperature within this range. Bread was best baked at from 180 to 240 deg., and the most economical temperatures were between 200 and 315 deg. For baking sponge cake the highest satisfactory temperature, 190 deg. C., was the most economical. It was considered feasible to reduce cooking to a more exact science.

In the discussion, the large and increasing number of electric ranges in use were made evident. By careful heat insulation and reduced losses the operating cost was being brought down close to gas. In Spokane, where gas was $1.15 per thousand cubic feet and electricity 10c. per kw.-hr. (base rate), there were about a hundred ranges. In St. Louis it was found that cooking current at 5c. per kw.-hr. equaled gas at $1.

Rates

One of the papers attracting the most interest was entitled "Class Rates for Light and Power Systems or Territories," by F. G. Baum, of San Francisco. Customers were divided into eight classes—(1) residence lighting, (2) commercial lighting and small power, (3) street lighting, (4) day power, (5) day and night power, (6) railway power, (7) irrigation, and (8) reclamation. The author had found close similarity in the load characteristics of such classes over large districts; therefore he proposed considering together all the customers of a system, irrespective of small local differences, and fixing the rates according to the averages for the classes, following the accepted scheme of making each class support its own investment and operating charges—in proportion to peak-loads demands and total energy drawn. The proposition was applied in detail for a hypothetical California case.

Miscellaneous Papers

The following papers completed the program:

1. "President's Address," by P. M. Lincoln, of Pittsburgh; a review of 15 years' progress and an argument that startling advances in efficiency of generators, motors, water wheels, engines and turbines were no longer possible.
2. "The Electric Strength of Air," by Prof. J. B. Whitehead, of Johns Hopkins University; a summary of theory for the behavior of air under electric stress.
3. "Form Factor and Its Significance," by Prof. F. Bedell, R. Bown and H. A. Pidgeon, of Cornell University; showing the futility of defining alternating-current wave shapes by "form factor"—ratio of root-mean-square to average value.
4. "Distortion Factors," by Prof. F. Bedell, R. Bown and C. L. Swisher, of Cornell University; numerical measures for the departure of an irregular wave from a pure sine wave.
5. "An Analytical and Graphical Solution for Nonsinusoidal Alternating Currents," by F. M. Mizushi, of Cornell University; modification of the usual expressions for alternating currents to make them hold true for nonsine waves.
6. "Classification of Alternating-Current Motors," by V. A. Fynn; "Classification of Electromagnetic Machinery," by F. Creedy.
7. "Phase Angle of Current Transformers," by C. L. Dawes, and "Calibration of Current Transformers by Mutual Inductances," by C. L. Fortescue; new laboratory and shop-testing equipment.
8. "The Induction Watt Hour Meter," by Prof. V. L. Hollister; an argument that induction meters did not behave as induction motors.
9. "Measurement of Dielectric Losses with the Cathode Ray Tube," by J. P. Minton, of Pittsfield; new apparatus for measuring the energy absorbed by insulation under various pressures.

Progress in Philadelphia's Rapid-Transit Plans

On July 1 Philadelphia City Councils enacted the necessary municipal legislation to authorize the placing under contract and the commencement of construction of the first section of the Broad St. subway and of a stretch of about 5 mi. of the Frankford elevated railway. The appropriation for the subway work is $3,000,000 and the appropriation for the elevated work is also $3,000,000.

The plans have yet to be passed upon by the Public Service Commission of the State of Pennsylvania, but in anticipation of its approval of them bids are now being invited to be opened Aug. 16 next, and prescribing that construction work shall be commenced Sept. 13 next.

The section of subway upon which the first work is to be done comprises the City Hall station section, which will be located diagonally under the west and south sides of City Hall and under the two lines of the present Market St. subway. The work will include underpinning City Hall and the Market St. subway structure.

The first contract on the Frankford elevated line which is expected to be awarded Aug. 16 will consist of the column foundations in Front St. from Callowhill St. to Kensington Ave., in Kensington Ave. to Frankford Ave., and in Frankford Ave. to Unity St., a distance of about 26,000 ft. Bids for the steel superstructure of the same stretch will be invited one month later. The reason for commencing the elevated work at Callowhill St. is that no operating agreement has yet been made, and therefore it is not yet known whether the line south of Callowhill St. will have to be connected to the Market St. system at Front and Arch St. for operation by the Philadelphia Rapid Transit Co., or whether it will have to be connected to a Chestnut St. subway for operation by some other company.

Completion of a Long Tunnel in France

The Mont d'Or tunnel through the Jura Mountains was opened to traffic on May 15, completing the new direct line for the Paris, Lyons & Mediterranean Ry. to the Simplon Tunnel route to Italy direct. The tunnel is 3¾ mi. in length and has a descending grade toward the Swiss end. It is laid with 96-lb. T-rails on creosoted oak ties. The trains will be handled by steam locomotives; but as the ventilation is said to be very effective, this is not objectionable. The tunnel was driven by Meyer compressed-air drills, four drills

being mounted on a heavy cross-bar wedged against the sides of the heading. This outfit was carried on a small four-wheel truck. The compressors were operated electrically by current from a Swiss hydro-electric plant. Compressed-air locomotives handled the construction cars.

The Swiss and French headings were started Nov. 14, 1910, and July 21, 1911, respectively. The work was in hard limestone, and in December, 1912, water was encountered, which increased from 660 to 1000 gal. per sec., drying up surface springs and a small river. A concrete bulkhead 20 ft. thick was built, the water being carried through pipes during its construction. The gate valves in these pipes were closed in January, 1913. The surface springs then flowed again. The damage done by the rush of water was repaired and a central drain built with a capacity of 220 gal. per sec. The pipes were then opened to drain off the water behind the bulkhead, and this was removed in February, 1913.

Other springs were encountered in April, 1913, when water burst in again and reached a flow of 2200 gal. per sec. The heading was again closed by a bulkhead and a side drift run parallel with it (and 65 ft. distant) to drain off the water. This drift encountered a great cave, which was utilized as a drainage well, into which the water was diverted. The heading was then reopened and the tunnel work proceeded. Wet spots in the stone lining of the roof were finally closed by forcing in grout under pressure.

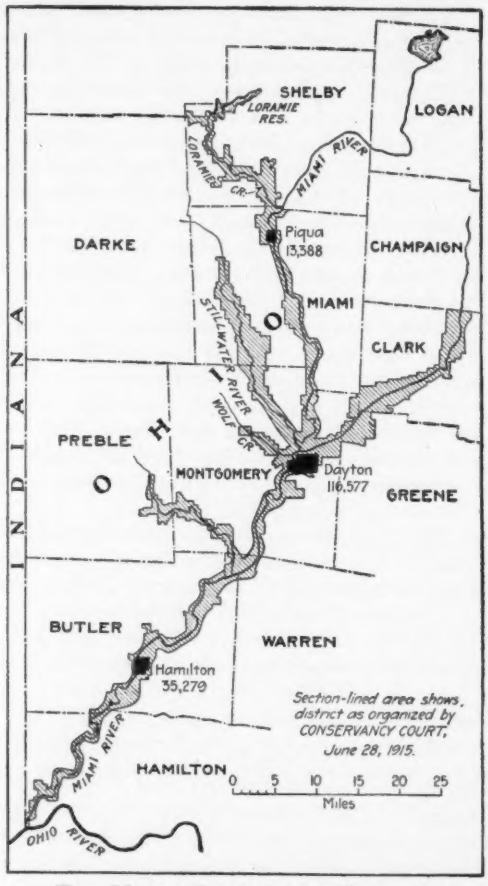
The new line between Frasne and Vallorbe is 15 mi. long and is double track, with maximum grades of 1.5 per cent. and a summit elevation of 2938 ft. It is thus much more favorable for fast and heavy traffic than the old circuitous single-track line by way of Pontarlier, which is 26 mi. long, with grades of 2.5 per cent. and a summit elevation of 3319 ft., and on which traffic in winter is subject to interruption by heavy snow.

Progress on the Miami Valley Flood-Prevention Work

On June 24 the Miami conservancy court convened at Dayton, Ohio, to consider a petition to organize a flood-protection district to be known as the Miami Conservancy District. The judges of Hamilton, Butler, Warren, Preble and Montgomery Counties voted to establish the district, and those of Miami, Greene, Shelby and Clark Counties dissented from the majority. The directors appointed by the conservancy court are Elward A. Deeds, of Dayton, Henry M. Allen, of Troy, and Gordon S. Rentschler, of Hamilton.

The court voted to exclude Logan County, owing to the fact that the detailed surveys and studies of various reservoir sites made by the Morgan Engineering Co. for the Flood-Prevention Committee have developed that the reservoir originally contemplated in Logan County is less efficient than at first supposed and that this reservoir can be compensated for at other sites at a substantial saving in cost. The accompanying map shows the district as planned.

The engineering work already completed under the supervision of Arthur E. Morgan, of the Morgan Engineering Co., includes a topographical survey of 240 sq.mi., property surveys of every piece of property in the proposed reservoirs, local, city and town surveys, relocation surveys for all steam and electric railways affected by the proposed reservoirs, the making of some 12,000 ft. of test borings on dam sites and many geological and hydrological studies and investigations. The preparation of construction plans is now in progress, about 20 engineers, draftsmen and computers being employed on this work. Sufficient progress has been made to determine the general character of the flood protection and prevention works, but there remain many details to be completed before actual construction work can proceed.



THE MIAMI CONSERVANCY DISTRICT

Bonds cannot be sold nor construction work started until the appraisal roll has been approved by the court. The total amount of money collected for the preliminary work to June 10 was nearly $478,000 and the total expenditures $301,000, leaving $176,000 to carry on this part of the work, which has included many local improvements.

Mr. Morgan has been made chief engineer of the work and will hereafter devote his entire time to it, instead of half time, as heretofore.

Failure of 17-Year-Old Concrete Roof Slab

One panel of the roof of a wing to the Gratiot School building in St. Louis, Mo., collapsed during a severe wind and rain storm on June 11, fortunately at a time when the building was unoccupied. The details of the collapse assume a certain importance because it has been errone-

ously reported as the failure of a long-established concrete building.

The wing in question was built in 1898. It comprises two stories and basement, with reinforced-concrete floor and roof slabs. There are brick bearing walls 24 ft. c. to c. which are spanned by I-beam joist 16 ft. c. to c. Into these I-beams frame 3x15-in. concrete beams spaced 18 in. c. to c. and each reinforced only with one 1-in. square deformed bar which extends simply into the concrete fireproofing around the I-beams and about 2 in. beyond the inside face of the end brick wall. The roof is a 2-in. unreinforced-concrete slab resting, without connection of any sort, on the concrete beams, and carrying a cinder concrete fill varying from 9 in. in thickness at the high point of the roof to zero at the drains.

One end panel, 16x24 ft. in plan, of this roof collapsed onto the second floor. The slab there was seriously damaged, but held under the impact of the 28-ton mass. The other panels of the floor and roof are intact, but damaged.

An examination of the roof after the failure showed that at the opposite end panel nearly all the concrete ribs had shear cracks at their supports, and in most cases were separated from the slab. Most of the ribs were badly deflected and had many diagonal tension cracks. The brick wall at the standing end panel was about ¾ in. out of plumb. It is assumed that the expansion of the cinder concrete fill pushed out the end walls and thus weakened the roof slab so that it was easily thrown down by the storm. At the same time it should be recognized that the whole construction is inherently weak and merited its long-delayed failure.

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Growth of American Merchant Marine—The tonnage increase of the American Merchant Marine up to May 1, 1915, amounted to 1019 vessels of 680,155 gross tons, according to a recent report of the Bureau of Navigation. This is a greater increase than has been recorded for any previous year. In 1907 there were added to the American registry 596,708 gross tons; previous to that the record had been held by the year 1855, when an increase of 586,102 gross tons was recorded.

The Hudson Bay Railway, from Le Pas (on the Canadian Northern Ry.) to Port Nelson, 420 mi., has the construction work well advanced. Work was started last spring with the grading at 290 mi. from Le Pas and tracklaying at 214 mi. The cantilever bridge over the Nelson River will be commenced and further work will be done on the terminals and harbor at Port Nelson. W. A. Bowder is Chief Engineer. The work is under the direction of the Department of Railways and Canals.

The Pan-American Scientific Congress is to be held in Washington, Dec. 27, 1915, to Jan. 8, 1916. The arrangements are in the hands of the Department of State. Hon. John Barrett, director-general of the Pan-American Union, is to act as secretary-general of the congress. The sessions will be held in the building of the Pan-American Union, which also will be the headquarters of the congress. Among the subjects which will be dealt with by the congress are meteorology and seismology, conservation, irrigation, engineering, mining and metallurgy, applied chemistry and transportation.

Freight Terminal Improvements at Chicago, proposed by the Atchison, Topeka & Santa Fé Ry., include a produce terminal at Wentworth Ave. and 16th St. Nothing definite has been decided, but the railway company has requested the city authorities to close some streets in a district where all the property is owned or controlled by the company, with the idea that if this is done the district can be utilized for terminal purposes. This would be for the railway company's own use. A project for a union terminal, warehouse and market for fruit and produce was noted in "Engineering News" of April 29.

Continuing Alaskan Railway Surveys—Thomas Riggs, Jr., member of the Alaskan Engineering Commission, left Seattle with his party May 6 and arrived at White Horse, Yukon Territory, May 11. As river navigation from White Horse is not open until June, he had the party transported overland, a distance of 135 mi., sending out the first wagon the day after he arrived. He reached Nenana on May 25, just 19 days after leaving Seattle, which was record time for that period of the year. There a camp had already been erected. On May 26 one of the party started up the Nenana River, and two days later the full party was in the field surveying.

The National Forest Reservation Commission has approved the purchase of lands in the White Mountains and the Southern Appalachians amounting to a total of 1,275,000 acres, the latest purchase of about 98,000 acres having been authorized on June 23. The prices paid for these latest purchases were $1.90 per acre for a block of 36,000 acres in North Carolina and $7.21 per acre for about 23,000 acres in New Hampshire. There were also acquired more than 13,000 acres at $2.70 per acre in the Natural Bridge area in Virginia. Practically all the land is mountainous and useful only for forest growth. Most of it has been cut over, but several large tracts have considerable timber on them.

Another Large Blast—A blast at the quarries of the Riverside Portland Cement Co. in California was described in "Engineering News," May 27, as the largest blast on record. The quantity of explosive used was 58 tons, nearly all 5% Hercules R. R. P.; and it was estimated that between 350,000 and 400,000 tons of rock was loosened. The "Engineering and Mining Journal" of May 29 describes a blast at the British Columbia Copper Co.'s Mother Lode mine, Greenwood, B. C., in 1913, in which 25 tons of 40% dynamite was used, and it was estimated that fully 400,000 tons of rock and ore were broken up. This, like the one at Riverside, was a strictly underground blast. There were 4830 holes loaded from interior shafts or tunnels.

The Panama Canal in its first eleven months of operation earned from tolls a little more than enough to pay the cost of its operation and maintenance. For the ten months ending Apr. 30, the tolls earned amounted to $3,338,000 and the expenses of operation and maintenance were $3,377,000, leaving a deficit of about $40,000. In May, however, the tolls earned amounted to $547,000, giving a sufficient margin above the expenses to wipe out the deficit and show a slight profit for the canal. It should be noted that in the cost of operation and maintenance is included the sum of $1,369,000 expended on dredging in the Gaillard cut. If this expense were charged to construction, the total cost of operating and maintaining the canal would be a little over $2,000,000 for its first year.

Hollow-Tile Walls are the subject of a specification recently adopted by the Hollow Building Tile Manufacturers' Association of America as a suggestion for municipal building codes. An absorption of 10 per cent. maximum and a minimum crushing strength of 3000 lb. per sq.in. net area are called for. The permissible loading is fixed at 200 lb. per sq.in. bearing members (continuous shell and web area.) The specification allows tile to be used in place of brick in all walls, including foundation walls, in the same gross thickness. For buildings outside the fire limits a minimum wall thickness of 6 in. (for one-story buildings) is specified. Copies of the specification may be secured from Charles T. Harris, secretary of the association, Engineers' Building, Cleveland, Ohio.

Reasons for Station Contracts in building the government railways in Alaska are given by William C. Edes, chairman of the Alaskan Engineering Commission, in a newspaper dispatch to the Boston "Transcript" from Seward, June 28. Mr. Edes is quoted as follows:

> We have been criticized for employing what is referred to as foreign labor and for cutting the so-called Alaska wage scale. The men gathered here to secure station work are above the average. Many come here from the railroads just completed in Canada, and we are fortunate to get such experienced help. The commission is handicapped by the Federal 8-hr. day law, because the season is only seven or eight months long and the men would prefer working longer hours for more pay. The wages paid are 37½c. an hour, which, after deducting $1 a day for board, leaves $2. With 24 hr. of daylight the men could work 10 hr. with ease. With this in mind, the commission is letting out contracts to groups of men and contractors so that they may work as long as they wish.

A Shortage of Potash Salts in the United States in 1914 was caused by the curtailment of imports consequent to the European War. Voluminous discussions as to the remedy for this condition have appeared, especially with regard to the fertilizer industry. A bulletin by W. C. Phelan, just published by the United States Geological Survey, discusses the situation and describes the work done in this country last year toward finding a domestic supply of potash salts. Potash salts imported for consumption in the United States in 1913 amounted to 612,514,916 lb., valued at $10,805,720. In 1913 nearly 1,000,000 lb. of potash fertilizer was used. On account of the field cost, investigations by the Survey have been limited to the drilling of a single deep well in the Black Rock Desert in Nevada, to general reconnaissance, and to tracing and checking reported discoveries or possibilities. The drilling uncovered no bed of potash or other salts.

Work on the Second Simplon Tunnel Continues in spite of the entrance of Italy into the war. The south end of the tunnel is on Italian territory, the north on Swiss territory; but at present work on the north end is discontinued, while progress at the south end was not interrupted when Italy declared war, except that the number of workmen was reduced by the call to colors, which required the recruiting of a new force. The May advance of each of the various elements of the work (crown heading, full section excavation, sidewall construction, arch construction) in the south end was about 875 ft.; heading work was prosecuted from a number of upraises. The total length of completed tunnel at the end of May was 15,232 ft. in the south end and 15,987 ft. in the north end, or a total of 31,219 ft., representing 48 per cent. of the final length of the second Simplon tunnel (65,026 ft.).—"Schweizerische Bauzeitung," June 12.

A Pavement Guarantee was sustained in a suit brought by the City of St. Paul against the Barber Asphalt Paving Co. to recover about $70,000. The company took 12 paving contracts in St. Paul some years ago, agreeing to make all necessary repairs on the pavement for a period of ten years. In October, 1912, the company refused to make repairs, claiming the damage to the pavement was due to leakage from gas mains, injury by street-railway tracks, excessive sprinkling of the streets, dirt in the gutters, insufficient crowning on some streets and the nature of the subsoil. The city therefore made the repairs and brought suit against the company for the amount expended. The U. S. District Court gave judgment in favor of the city, and that judgment has now been affirmed by the Circuit Court of Appeals, which holds that the company was bound to have had knowledge of the conditions likely to cause injury to the pavements when it entered into the contract.

Lock No. 4 at the Soo Canal has the excavation completed by the MacArthur Brothers Co., of Chicago, and work will soon be commenced by the contractors for the masonry, the Oscar Daniels Co., of Chicago. The lock is parallel with and a duplicate of Lock No. 3, which was described fully in "Engineering News," Mar. 5 and Apr. 23, 1914. It will be 1350x80 ft., with from 24½ to 25 ft. of water on the sills. The filling culverts will extend under the floor for the full length of the lock and the emptying culverts will extend from its lower end to discharge beyond the lower gates. These culverts will have butterfly valves operated by hydraulic cylinders. The steel lock gates will be operated by electric winding engines. All the work of construction and operation is under the direction of Louis C. Sabin, General Superintendent of the St. Mary's Falls Canal. Lieut.-Col. Mason M. Patrick, Corps of Engineers, U. S. A., is in charge of the district which includes the Soo canals and locks.

A Patent Decision on electric car-lighting systems, recently delivered by the United States Circuit Court of Appeals for the Second District, in the case of the Safety Car Heating & Lighting Co. versus the United States Light & Heating Co., sustains the decision of the lower court upholding the first eight claims of patent No. 747,686 to J. L. Creveling, dated Dec. 22, 1903, and owned by the Safety company. The patent covers an electrical regulating system by which the current delivered by the generator, which is driven from the car axle at rates varying with the speed of the train, is maintained at substantially constant voltage. The court's decision reviews the methods of regulation adopted by various other inventors and awards priority to Creveling. A decision in favor of the Safety company was also entered on June 1 by Judge Hazel in the United States District Court of Western New York in a suit against the Goulds Coupler Co. for infringement of patent No. 1,070,080, to H. G. Thompson, dated Aug. 12, 1913. The judge held that claims 4, 9, 10, 11, 12 and 13 of the Thompson patent were infringed by the Simplex system, made by the Goulds Coupler Co.

An Automatic Train-Stop and Cab Signal system was given a practical trial on the Cincinnati Southern Ry. a few weeks ago. The track installation has been made on 18 miles of the southbound track between Erlanger and Crittenden, Ky., this being a double-track line equipped with automatic three-position block signals. At each block a track contact and a 6-ft. bar outside of and a little above the track rail are located 600 ft. in advance of the fixed signal. The tender carries shoes which engage the contact device, and circuit-breakers which are engaged by the bar. The engine is equipped with a three-light signal and a speed recorder driven from one of the truck axles. When the block signal is at "proceed," the cab device shows a white light while the train is running through the block. If the signal is at the "proceed under control" position, a green light is shown, while a "stop" signal causes a red light to be shown in the cab. If a train passes a "stop" signal, the circuit closes an air valve and the brakes are applied. When it has come to a full stop, the needle of the speed recorder closes a circuit, and the train can proceed no faster than 10 m.p.h. to the next signal. The circuit breaker is to insure that no cab signal is carried through from one block to the next, but is actuated definitely at each block. A special feature of the system is its speed-control arrangement. If the train exceeds 25 m.p.h. while in a block governed by a caution signal, the brakes will be applied, but will be released automatically when the speed has been reduced within this limit. By the same means the maximum speed of the train can be kept within any desired limit. This control is secured by contacts made with the indicating needle of the speed recorder. The system is controlled by the Julian-Beggs Signal Co., of Terre Haute, Ind.

PERSONALS

Mr. B. H. Klyce, Assoc. M. Am. Soc. C. E., was recently appointed City Engineer of Miami, Fla.

Mr. William B. Winslow, a lawyer of New York City, has been appointed Receiver for the Florida Railway Co.

Mr. A. K. Nicolaysen, Assistant City Engineer in Charge of Bridge Construction of San Antonio, Tex., has resigned from the city's service.

Mr. W. H. Keller has been appointed Master Mechanic of the Baltimore & Ohio Southwestern R.R., with headquarters at Cincinnati, Ohio.

Mr. Arthur S. Piers has been appointed Manager of the Real Estate Department of the Canadian Pacific Ry., with headquarters at Montreal.

Mr. E. M. Turner, for the past 12 years Assistant City Engineer of Shreveport, La., has been elected City Engineer, to succeed Mr. George R. Wilson, resigned.

Mr. Frank E. Davidson, M. Am. Soc. C. E., of Patterson & Davidson, Engineers and Architects, Chicago, Ill., has been elected President of the Illinois Society of Architects.

Lieut.-Col. Jay J. Morrow, Corps of Engineers, U. S. A., has been relieved from duty at Portland, Ore., to take effect Aug. 1, 1915. He is to go to the Panama Canal Zone for duty under the Governor.

Mr. H. M. Montgomery, Chairman of the Chicago Section of the American Society of Mechanical Engineers, has been elected Vice-President and Director of the Powdered Coal Engineering & Equipment Co., of Chicago.

Mr. R. L. Squier, Deputy County Engineer of Champaign County, Ohio, recently passed the bar examination at Columbus, Ohio, and will be admitted to practice. Mr. Squier is a graduate civil engineer and will continue his practice as such.

Mr. Guy Nevil, who has been connected with the City Engineering Department of San Antonio, Tex., has resigned and expects to sail for Spain in the near future, where he will be connected with one of the Pearson Engineering Co.'s projects.

Messrs. H. C. Trexler, Malcolm Gross, City Solicitor; David Williams, Editor of the "Labor Herald;" E. J. Lumley, President of the Chamber of Commerce; F. W. Weil and S. A. Weishampel have been appointed members of the City Planning Commission of Allentown, Penn.

Mr. Hurlbut S. Jacoby, formerly Chief Engineer of the Standard Steel Construction Co., of Welland, Ont., now Instructor in Civil Engineering at Pennsylvania State College, will spend the summer as Associate Engineer with the Samuel Austin & Son Co., of Cleveland, Ohio.

Mr. E. F. Booth, Assistant City Engineer of Youngstown, Ohio, has resigned and will enter into partnership with R. C. Shook, local contractor, who has been awarded the contract to grade and build the bridge foundations for the Fredericksburg road at the Milton Reservoir site.

Mr. H. E. Bilger, M. Am. Soc. C. E., Road Engineer, Illinois State Highway Department, Springfield, Ill., was granted the degree of Master of Science, pro merito, by Bucknell University, on June 16. Mr. Bilger is a graduate of both Bucknell University and the University of Missouri.

Mr. Thomas J. McMaster, who for the last 30 years has acted in the capacity of Sales Engineer for a number of concerns in the West, has been made District Manager of the Diamond Power Specialty Co., of Detroit, Mich. His headquarters will be at 809 Ashland Block, Chicago, Ill.

Mr. Frank R. Farnham, previously with the Make-It-Pay Department of the Hill Publishing Co. and later in charge of the Copy Service Department of the McGraw Publishing Co.,

has joined the Staff of Walter B. Snow, Publicity Engineer and Advertising Agent, 136 Federal St., Boston, Mass.

Mr. M. L. Scannell, Manager of Water-Works of Tiffin, Ohio, has resigned, after 37 years of service as an officer and director of the company. He will be succeeded by Mr. C. H. Witter, who has been for some time employed as Superintendent of Construction and Operation by the company.

Mr. L. R. Jorgensen, M. Am. Soc. C. E., who for many years has been associated with the engineering firm of F. G. Baum & Co., San Francisco, Calif., has severed his connections with that company and will engage in a general electric and hydraulic engineering practice at 1404 Chronicle Bldg., San Francisco. Mr. L. Jorgensen is the inventor and patentee of the constant-angle multiple-radius dam. He will make a specialty of the design of arch dams.

Prof. Charles M. Spofford, M. Am. Soc. C. E., of the firm of Fay, Spofford & Thorndike, Consulting Engineers, Boston, Mass., and Head of the Department of Civil and Sanitary Engineering of the Massachusetts Institute of Technology, has been appointed by Governor Walsh, of Massachusetts, one of the commissioners to investigate the subject of terminal facilities and the improvement of facilities for the transportation of freight in the Metropolitan District.

The following changes have been announced by the Wabash R.R.; Mr. S. E. Cotter, General Superintendent, at St. Louis, has been appointed General Manager, and will continue to have headquarters at St. Louis; Mr. J. W. Jones, Superintendent of Terminals, at St. Louis, has been appointed Superintendent of the Western division and will have headquarters at Moberly, and Mr. C. E. Ocheltree, Assistant Superintendent, at Forrest, Ill., will succeed Mr. Jones.

Lieut.-Col. C. W. P. Ramsay, Assoc. M. Can. Soc. C. E., formerly Engineer of Construction of the Eastern lines of the Canadian Pacific Ry., is now in command of the Canadian Overseas Railway Construction Corps of the Allies' armies in France. He entered the service of the Canadian Pacific in 1898. In 1912 he was appointed Engineer of Construction for Eastern lines, and has been closely identified with the construction of all the company's new lines since that time, notably the building of the lake shore line from Montreal to Toronto.

Mr. J. F. Trumbull, Chief Clerk to the Chief Engineer of the New York, New Haven & Hartford R.R., has been appointed Chief Engineer of the Public Utilities Commission of Connecticut at a salary of $3000 a year. Mr. Trumbull is a graduate of Sheffield Scientific School, Yale University, and since graduation has been engaged in steam and electric railway engineering work, having been employed by the New York, New Haven & Hartford R.R., the International R.R. of Mexico and the Connecticut Co. Mr. Trumbull passed the civil-service examination for the position of Chief Engineer with a rating of 97.5 per cent.

Mr. Edward F. Vincent, M. Am. Soc. C. E., Assistant Chief Engineer of the Colorado & Southern Ry., has been appointed Chief Engineer, with headquarters at Denver. Mr. Vincent was born at Maquoketa, Iowa, in 1863. He is a graduate of the State University of Iowa. In 1886 he entered the service of the Union Pacific Ry. as draftsman and field assistant. The following year he was engineer on construction of the State Capitol Building at Denver, returning the next year to the Union Pacific. Later he was connected with the U. S. Geographical Survey, in charge of a party on reservoir sites near Leadville, Colo., and was for a time Principal Assistant Engineer of the Board of Public Works of Denver. He entered the service of the Colorado & Southern R.R. in 1906.

The Pennsylvania Company announces the following changes made in its operating department: Mr. S. B. Robertson, Superintendent of the Erie & Ashtabula division, at Newcastle, Penn., has been transferred to Cleveland as Superintendent of the Cleveland and Pittsburgh division, succeeding Mr. I. W. Geer, who has been promoted to General Superintendent of the Central System, with headquarters at Toledo. Mr. W. M. Wardop, Superintendent of the Fort Wayne division, will succeed Mr. Robertson, and Mr. Otto Schroll, Superintendent of the Toledo division, will succeed Mr. Wardop. Mr. Paul Jones, Superintendent of the Zanesville division, will succeed Mr. Schroll, and F. J. Stimson is promoted to Superintendent of the Zanesville division, succeeding Mr. Jones.

Mr. E. F. Saxton has been appointed Assistant Director of the Department of Wharves, Docks and Ferries of Philadelphia, Penn. For the last three years Mr. Saxton has been Acting Supervising Engineer of the Permit Bureau of the Philadelphia Bureau of Highways, and since the death of M. M. Coppuck has been Secretary of the Board of Highway Supervisors. He is a graduate of Lehigh University and of the New York University Law School. Mr. Saxton was connected with the Engineering Department of the New York Central & Hudson River R.R. during the construction of the Park Ave. improvements, and has had a varied experience in bridge and railway construction. Subsequently he was connected with the Chicago & Northwestern Ry. and the Mexican Central R.R.

OBITUARY

Aaron Ward, a well-known contractor of Camden, N. J., died on June 28 at the Cooper Hospital in that city in his 81st year. Mr. Ward was born in Camden and most of his life was engaged in the contracting business.

ENGINEERING SOCIETIES

COMING MEETINGS

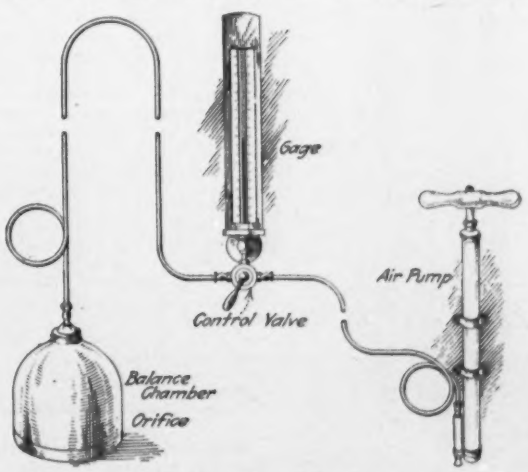
AMERICAN INSTITUTE OF CHEMICAL ENGINEERS.
Aug. 16-18 and 25-28. Annual meeting in Los Angeles, Calif., Aug. 16-18; in San Francisco, Aug. 25-28. Secy., J. C. Olsen, Cooper Union, New York City.

INTERNATIONAL ASSOCIATION OF MUNICIPAL ELECTRICIANS.
Aug. 24-27. Annual convention in Cincinnati. Secy., Clarence R. George, Houston, Tex

Appliances and Materials

"Pneumercator" Depth Gage

A device, called the "Pneumercator," for measuring the depth and volume of tanks, etc., and accurately registering the measurement at any convenient place has been invented by H. S. Parks, of Philadelphia, and is being introduced by the Pneumercator Co., 118 Liberty St., New York City. The various parts of the system, as shown in the sketch herewith, include a balance chamber, a mercury gage, a three-way control valve and an air pump—all properly connected by small flexible piping. The balance chamber is placed inside the tank, with its orifice at some point which it is desired to represent by the zero on the gage scale. With the chamber connected to the gage the rising liquid traps and compresses the air in the chamber and pipe line, but the gage does not register accurately, the error being proportional to the height of liquid above the orifice. To set the gage accurately the valve is changed to connect the chamber with the pump, which is given a few strokes to expel any liquid above the orifice. The air pressure remaining after the pump is disconnected and the pipe dead-ended (by turning the control



TANK-LEVEL GAGE

valve), balances the liquid column outside the chamber. Now by connecting the chamber to the gage the correct height is shown, and thereafter the device registers properly—there being only an inappreciable compression and displacement of the air. The chamber and piping are blown out once in a while to guard against slow leaks.

The gage may be located above or below the body of liquid measured. Where the tank is under greater than atmos-

pheric pressure the upper part of it is connected to the top of the mercury tube.

A trial installation has been placed in the fuel-oil tanks of the United States battleship "New York, and is reported to give oil consumption with an accuracy not before possible. By placing fore and aft balance chambers in casings connected to the sea, the draft and trim of a vessel can be registered. Several such installations on commercial boats have been reported.

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Backfilling and Pipe-Handling Machine

A portable gasoline hoist for operating a derrick and scraper and intended specially for trench work is now built by the Waterloo Cement Machinery Corporation, of Waterloo, Iowa. The bedplate carying the hoist and engine is pivoted to a wagon bed so that it can swing horizontally. The machine weighs about 2600 lb. and is equipped with a 4½-hp. engine.

For backfilling, the machine travels along the clear side of the trench, and its cable is attached to a scraper handled by a man on the opposite side. The machine hauls the scraper across the spoil bank, dragging a load into the trench, and the man pulls it back for another haul. The swinging of the machine enables it to backfill a considerable length of trench at each move. One contractor reports having backfilled 300 cu.yd. in 8 hr., at 2c. per yd., with the machine, while another has used it (with two men) in place of a gang of 20 shovelers.

To move the machine, the hoist is locked in line across the wagon bed and a cable is led through a sheave anchored ahead and given a turn around the hoist's niggerhead. The main cable can be used also in unloading pipe from cars and lowering it into the trench, and for miscellaneous service on construction work.

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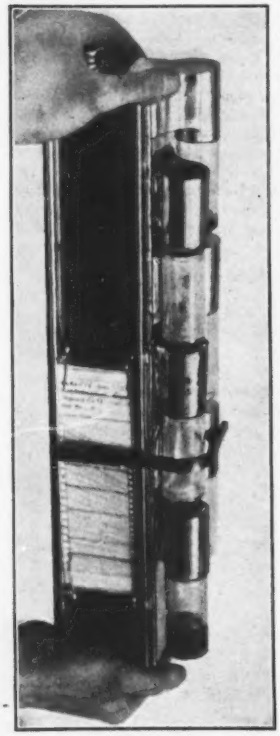
Field Sand-Testing Device

The device shown in the accompanying view is designed for making granulometric analyses of sand and is intended mainly to be used in rapid field investigations of concrete sands. It consists of a metal box containing a ladder of graduated screens soldered into the case on planes which slope down to openings into the glass tubes shown in the view. Sand mixed with a little water is poured in the top of the case and, by agitating, is passed down through the successive screens, the selections being passed out into the glass tubes. In this way the amounts of the various gradations of sizes appear each in its proper glass tube. Measurement is then made by marking off an a card which fits in a frame that slides on rods up one side of the case the exact amount in each grade, adding graphically each amount to the one previously measured. The exact percentages of the various amounts may then be read from the interpolating lines permanently ruled on the card.



JOHNSON-CHAPMAN SAND TESTER

The device was invented by Nathan C. Johnson and Cloyd M. Chapman and is being marketed by the Concrete Appliances Co., Inc., Singer Building, New York City.

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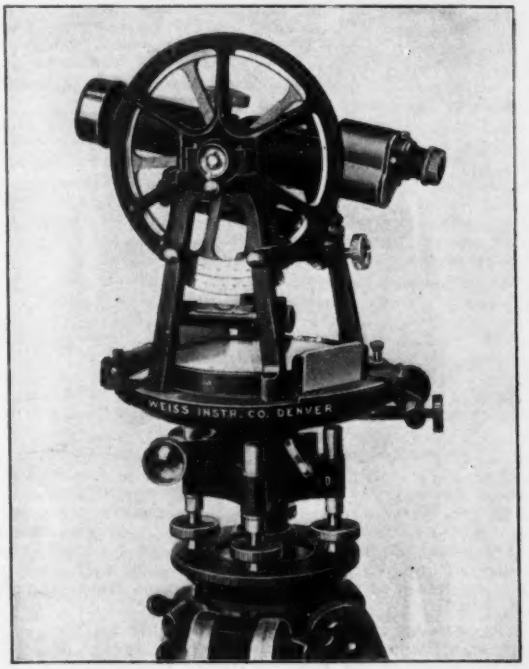
Transit with Poro Prism Telescope

The accompanying illustration shows a new type of engineers's and surveyor's transit recently patented and put on the market by the Weiss Instrument Co., of Denver, Colo. With the Poro prism type of telescope shown, the image is erected in the direct path from the objective to the eye-piece, permitting the use of an astronomical eye-piece, which gives greater light and field than an erecting eye-piece. Other advantages claimed are that the telescope is much shorter for the same magnifying power, which indirectly means lower standards and greater rigidity and stability. The Poro prism telescope also makes it possible to use different eye-pieces, which are made in 15-, 20- and 25-diameter sizes. The maker guarantees this type of telescope to maintain its line of collimation under all conditions.

The telescope is 8 in. long for a magnifying power of 20 diameters, against 12 in. for the regulation transit telescope. The field is 1° 45′ with an aperture of 1⅛ in. The standards are 4¾ in. high from plate to center of telescope bearings.



TRANSIT WITH PORO PRISM TELESCOPE

The instrument described and illustrated weighs 10 lb. The light mountain transit of the same type has a telescope only 6½ in. long, a magnifying power of 17 diameters, a field of 2° 15′, an aperture of 1⅛ in., and weighs 9 lb.

* * *

Non-Slip Ladder Shoe

An anti-slipping shoe for ladders has been put on the market by the American Mason Safety Tread Co., 50 Church St., New York City. The shoe consists of a cast plate, with triangular lugs coming up on either side of the ladder leg. The bottom is faced with carborundum grit, and it is claimed that the shoe will safely grip on the smoothest concrete or steel floor. A bolt passes through the lugs and leg so that the shoe is pivoted on the end of the string-piece; this permits the face of the shoe to lie flat against the floor regardless of the angularity of the ladder.

* * *

Portable Rivet Forge

A portable rivet forge has been designed by the Mahr Manufacturing Co., Minneapolis, Minn., for general use in shops and on construction work. The forge uses oil fuel, and an 18-gal. tank forms part of a welded truck frame. This is on three wheels, the front one being in a pivoted fork carrying the pull handle. Steel-angle legs support a firebrick-lined steel furnace with combustion and heating spaces. A chain bail on the furnace hood allows lifting the machine by a crane. Compressed air at from 10 to 125 lb. pressure is required.

* * *

Pressed Asbestos Products

Material for a wide variety of industrial and commercial fittings' and composed largely of asbestos and binder, is being marketed by the Pressed Asbestos Products Co., 451 W. 19th St., New York City. The material, called "Papco," is furnished in densities varying from that of soft wood to that of stone; it is an electrical insulator and can be produced to withstand temperatures up to 2600 deg. F. It has been employed for handles on hot utensils, for bowling balls, table tops, switchboard panels, rheostat bases, and sanitary seats. For many such services it is claimed to serve as well as porcelain, but not to be cold to the touch.